SMIP02

SMIP02 SEMINAR ON
UTILIZATION OF STRONG-MOTION DATA

Los Angeles, California
May 2, 2002

PROCEEDINGS

Sponsored by

California Strong Motion Instrumentation Program
California Geological Survey
California Department of Conservation

Supported in Part by

California Seismic Safety Commission
Federal Emergency Management Agency

The California Strong Motion Instrumentation Program (CSMIP) is a program within the Division of Mines and Geology (now also known as the California Geological Survey) of the California Department of Conservation and is advised by the Strong Motion Instrumentation Advisory Committee (SMIAC), a committee of the California Seismic Safety Commission. Current program funding is provided by an assessment on construction costs for building permits issued by cities and counties in California, with additional funding from the California Department of Transportation, the Office of Statewide Health Planning and Development, and the California Department of Water Resources.

In 1997, a joint project, TriNet, between CSMIP, Caltech and USGS at Pasadena was funded by the Federal Emergency Management Agency (FEMA) through the California Office of Emergency Services (OES). The goals of the project are to record and rapidly communicate ground shaking information in southern California, and to analyze the data for the improvement of seismic codes and standards. TriNet produces a ShakeMap of ground shaking , based on shaking recorded by stations in the network, within minutes following an earthquake. The ShakeMap will identify areas of greatest potential for use by OES and other emergency response personnel in the event of a damaging earthquake.

In northern California, CSMIP is partnering with UC Berkeley and USGS at Menlo Park. In July 2001, the California Office of Emergency Response started to obtain funding for the California Integrated Seismic Network (CISN), a statewide system that includes the TriNet system. The CISN will improve seismic instrumentation and provide statewide ground shaking intensity maps. It will also distribute and archive strong-motion records of engineering interest and seismological data for all recorded earthquakes, and provide training for users.

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PREFACE

The California Strong Motion Instrumentation Program (CSMIP) in the Division of Mines and Geology (now also known as the California Geological Survey) of the California Department of Conservation established a Data Interpretation Project in 1989. Each year the CSMIP funds several data interpretation contracts for the analysis and utilization of strong-motion data. The primary objectives of the Data Interpretation Project are to further the understanding of strong ground shaking and the response of structures, and to increase the utilization of strong-motion data in improving post-earthquake response, seismic code provisions and design practices.

As part of the Data Interpretation Project, CSMIP holds annual seminars to transfer recent research findings on strong-motion data to practicing seismic design professionals, earth scientists and post-earthquake response personnel. The purpose of the annual seminar is to provide information that will be useful immediately in seismic design practice and post-earthquake response, and in the longer term, in the improvement of seismic design codes and practices. The SMIP02 Seminar is the 13th in a series of annual seminars. Last year, the SMIP01 Seminar scheduled to be held in Los Angeles on September 12, 2001 was not held due to the tragic events of September 11. However, SMIP published the proceedings for the SMIP01 Seminar and results of the two CSMIP-funded projects will be presented at the SMIP02 Seminar.

In this seminar, the sessions in the morning will focus on the ATC-54 Guidelines for Using Strong-Motion Data and ShakeMap in Post-Earthquake Response. Before the three presentations on the ATC-54 Guidelines, we have invited Dr. Wald to give an introduction on the ShakeMap. Dr. Bozorgnia will present the results from his CSMIP-funded studies on ground shaking parameters for post-earthquake applications. At the luncheon, Mr. Ellis Stanley, General Manager of the Los Angeles City Emergency Preparedness Department, is the keynote speaker.

The sessions in the afternoon will include presentations by investigators of three CMIP-funded projects currently under way. These projects are scheduled to be completed by the end of 2002, so the investigators can only present preliminary or interim results. The final results will be presented at the next year's seminar (SMIP03) and in their final reports. In addition to CSMIP-funded projects, there will be presentations on the Internet Quick Report from the CISN Engineering Data Center and summaries of the two workshops recently held by the Consortium of Organizations for Strong-Motion Observation Systems (COSMOS).

Moh J. Huang
Data Interpretation Project Manager

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Jennifer Swift and J. Carl Stepp

SMIP02 SEMINAR ON UTILIZATION OF STRONG-MOTION DATA

Hyatt Regency Hotel, Los Angeles, California
May 2, 2002

FINAL PROGRAM

8:00 am **Registration**

9:00 am **Welcoming Remarks**
Bruce Clark, Chair, California Seismic Safety Commission
Anthony Shakal, Supervising Geologist, California Geological Survey

9:10 am **Introductory Remarks**
*Moh Huang***,** California Strong Motion Instrumentation Program

SESSION I

Moderator: *Marshall Lew*, Law Crandall, SMIAC

9:20 am **ShakeMap: Its Role in Pre-Earthquake Planning and Post-Earthquake Response and Information**
David Wald, *C. Bruce Worden*, *Vincent Quitoriano*, USGS; and *Jame Goltz*, OES

9:35 am **Guidelines for Utilizing Strong-Motion and ShakeMap Data in Post-Earthquake Response: An Overview** (***ATC-54, Part 1***)
Christopher Rojahn, Applied Technology Council; *Craig Comartin*, Comartin-Reis; and *Stephanie King*, Hart-Weidlinger

9:45 am **Guidelines for Utilizing ShakeMaps for Post-Earthquake Response** (***ATC-54, Part 2***)
Stephanie King, Hart-Weidlinger; *Craig Comartin* and *Evan Reis*, Comartin-Reis; *Sarah Nathe*, UC Berkeley; and *Maurice Power*, Geomatrix Consultants

10:10 am **Questions and Answers for Session I**

10:30 am Break

SESSION II

Moderator: *Chris Poland*, Degenkolb Engineers, SMIAC

10:50 am **Guidelines for Utilizing Strong-Motion Data for Post-Earthquake Evaluation of Structures** ***(ATC-54, Part 3)***
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<u>*Yousef Bozorgnia*</u>, Applied Technology & Science; and *Vitelmo Bertero*, UC Berkeley

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Speaker: <u>*Ellis Stanley*</u>**,** General Manager, Los Angeles City Emergency Preparedness Department

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Moderator: *Vern Persson*, SMIAC

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2:15 pm **Analysis of Pacoima Dam Response Using Recently Recorded Seismic Motions: Report on Progress**
<u>*John Hall*</u> and *Steven Alves*, Caltech

2:40 pm **Questions and Answers for Session III**

2:50 pm Break

SESSION IV

Moderator: *Wilfred Iwan*, Caltech, SMIAC

3:10 pm **Rapid Dissemination of Strong-Motion Data via the Internet Quick Report at the CISN Engineering Data Center**
<u>*Kuo-Wan Lin*</u>, *Anthony Shakal* and *Moh Huang*, CSMIP/CGS; and *Chris Stephens* and *Woody Savage*, NSMP/USGS

3:35 pm **Development of An Engineering Model of the Amplitude and Duration Effects of Basin Generated Surface Waves**
<u>*Paul Somerville*</u>, *Nancy Collins*, *Robert Graves* and *Arben Pitarka*, URS Group

4:00 pm **Results from COSMOS Building Instrumentation and Geotechnical Data Workshops**
<u>*J. Carl Stepp*</u>, COSMOS

4:15 pm **Questions and Answers for Session IV**

4:20 pm **Closing Remarks**

Members of the Strong Motion Instrumentation Advisory Committee

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Ashok Patwardhan, Chair, Geotechnical Consultant
Bruce Bolt, UC Berkeley
Anil Chopra, UC Berkeley
C. Allin Corbnell, Stanford University
Wilfred Iwan, California Institute of Technology
Jerve Jones, Peck/Jones Construction Corp.
Vern Persson, DWR Div. of Safety of Dams (retired)
Chris Poland, Degenkolb Engineers
Ray Zelinski, Caltrans
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SHAKEMAP: ITS ROLE IN PRE-EARTHQUAKE PLANNING AND POST-EARTHQUAKE RESPONSE AND INFORMATION

David J. Wald, C. Bruce Worden, and Vincent Quitoriano

United States Geological Survey
Pasadena, California

James Goltz

California Governor's Office of Emergency Services
Pasadena, California

Abstract

ShakeMap was designed primarily as a rapid response tool to portray the extent and variation of ground shaking throughout southern California immediately following significant earthquakes. The system now runs throughout California as well as in the Salt Lake City and Seattle areas, and it is being expanded to populated, seismically active regions nationally as resources permit. For rapid response, ShakeMap ground motion values are used for emergency response and loss estimation, assessment of damage to the lifeline and utility networks, and for providing information to the general public. However, ShakeMap can also be used as a pre-earthquake planning tool by generating ground motion estimates for a suite of potential earthquake scenarios. Estimates based on earthquake scenarios can provide a firm basis for loss estimation on a regional scale as well as provide utilities and other users a means of evaluating their emergency response capabilities. This paper will examine the practical applications of ShakeMap in emergency response, engineering, planning, training, and public education considering both current and future applications.

Introduction

ShakeMap™ is one of the first and, for emergency management, most significant products of the TriNet project. The TriNet project is named for the three organizations that have collaborated to build a new digital seismic network in southern California: the California Institute of Technology, the State of California, Division of Mines and Geology (now the California Geological Survey, CGS) and the United States Geological Survey. The five-year project to develop, install, and operate the network was completed at the beginning of 2002. In addition to ShakeMap, the "real-time information" products from TriNet are direct results of the new digital seismic and strong motion networks and include the rapid broadcast and web posting of accurate and reliable information on magnitude, location, and aftershock distribution. For more information on TriNet, see both Mori *et al.* (1998) and Hauksson et al. (2000).

As TriNet funding from FEMA ended at the beginning of 2002, TriNet continued, but under the auspices of the California Integrated Seismic Network (CISN) as a region of the Advanced National Seismic System (ANSS). Funding from the USGS continues, and the State of California Office of Emergency Services (OES) is providing additional funds. CISN statewide coordination includes the three TriNet partners as well as the Menlo Park office of the USGS and the Berkeley Seismological Laboratory (BSL) at the University of California at Berkeley and the OES. Figure 1 shows the distribution of seismic stations for the CISN. As of April 2002, there were approximately 200 real-time stations online and nearly 400 CGS dialup stations statewide in the CISN.

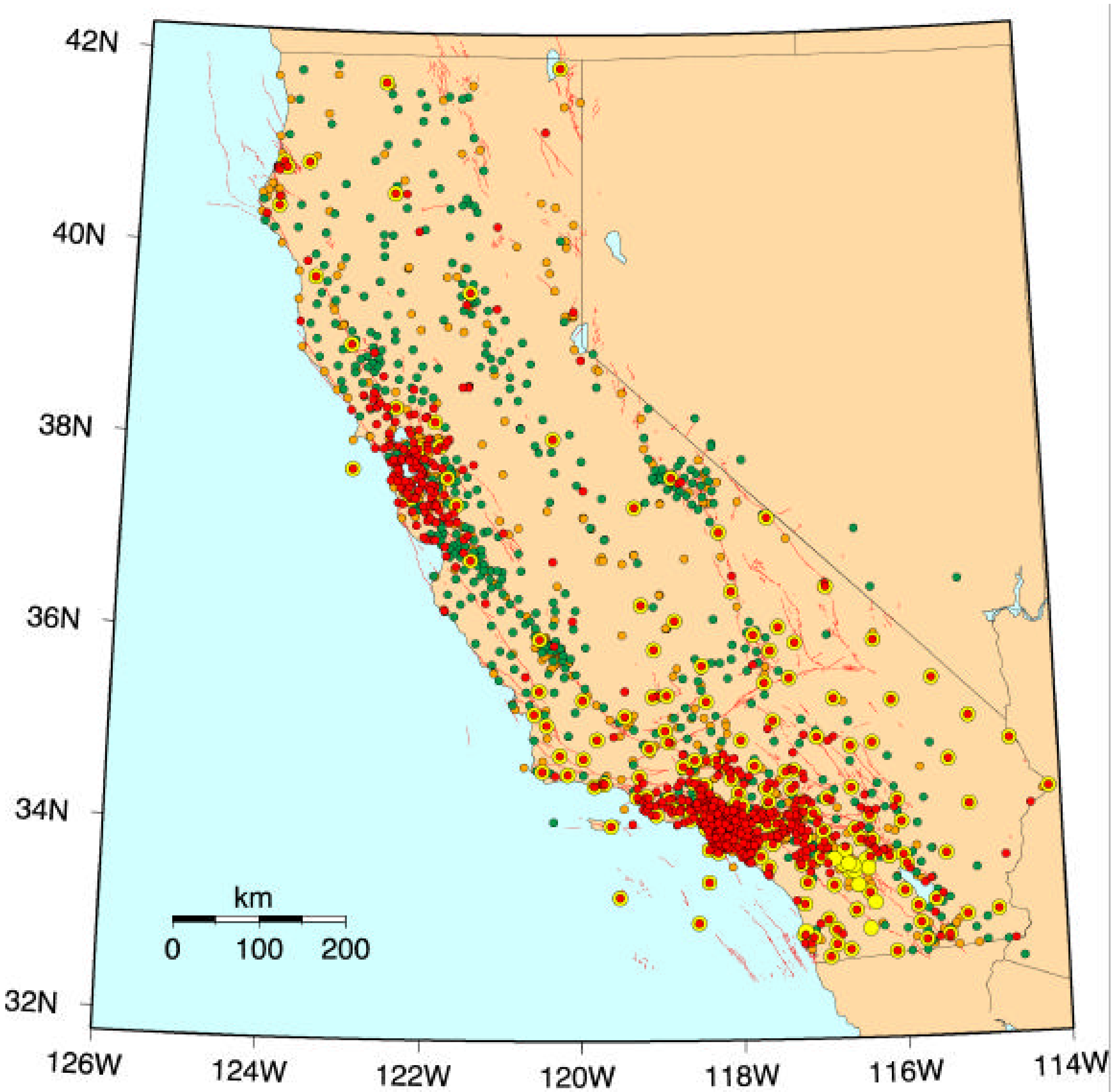


Figure 1. CISN seismic station map. Circles are broadband (yellow), real-time and dialup ShakeMap quality strong-motion (red), non-ShakeMap quality (orange), and weak motion velocity (green) seismometers. Figure courtesy of D. Oppenheimer.

A key requirement of the OES funding for CISN is the integration of both northern and southern California seismic networks into a unified system with statewide reporting of earthquakes and built in redundancy, including backup generation of ShakeMap. Currently however, ShakeMap operates and is reported separately in southern and northern California.

Ongoing development of ShakeMap is under the auspices of the U.S. Geological Survey's Advanced National Seismic System (ANSS). Under this program, ShakeMap now runs in southern and northern California, as well as the Seattle and Salt Lake City areas. It will be available in other seismically active regions of the country as sufficient numbers of real-time strong motion stations are installed as outlined in the ANSS strategic plan (U.S. Geological Survey, 2000), if funds become available.

Making a ShakeMap

ShakeMap relies primarily on observed ground motion shaking levels determined rapidly from free-field strong motion seismic instruments. Within the first minute following an earthquake, ground-motion parameters are available from the real-time component of the network and within several minutes most of the important near-source dial-up stations contribute. Initial maps are made with just the real-time (continuously-telemetered) component of CISN, but they are updated automatically as more data are acquired. Parametric data from the stations include peak ground acceleration (PGA), peak ground velocity (PGV), and peak response spectral amplitudes (at 0.3 sec, 1 sec, and 3 sec). At the same time, maps of instrumental intensity are generated through relationships between recorded ground-motion parameters and expected shaking intensity values (Wald *et al.*, 1999a) developed specifically for ShakeMap. Production of the maps is automatic, triggered by any significant earthquake in California (see Wald *et al.*, 1999b, for more details).

Figure 2 presents an example of the data and processing that produce a ShakeMap, in this case for the 1999 Hector Mine, California earthquake. The first panel of the figure shows an sample of the variety of site soil conditions in southern California, and the variability of seismic station distribution depending on proximity to urban areas. Estimation of shaking over the entire affected region is obtained by the spatial interpolation at sites in between the measured ground motions with geologically based frequency- and amplitude-dependent site corrections. Site conditions become an important part of the ShakeMap pattern, particularly where the network is sparse and fewer data are available. We use CGS maps of National Earthquake Hazard Reduction Program (NEHRP) classification site conditions as the basis for our site corrections. These site condition maps have coverage throughout the state at 1:250,000 scale (Wills *et al.*, 2000).

Since this event was centered in a remote area there were few near-fault seismic stations. Initially, ShakeMap ground motions in the near-source region, and other areas without seismic stations, were estimated using an empirical ground motion regression with distance measured from the epicentral location. Later, as information about fault dimensions became available (in the form of aftershocks, source rupture models, and observed surface slip), the fault location and rupture dimensions were employed as the basis for ground motion estimation in the near-source region. The second panel shows the results of interpolated data and estimated amplitudes to produce a map of peak ground motions.

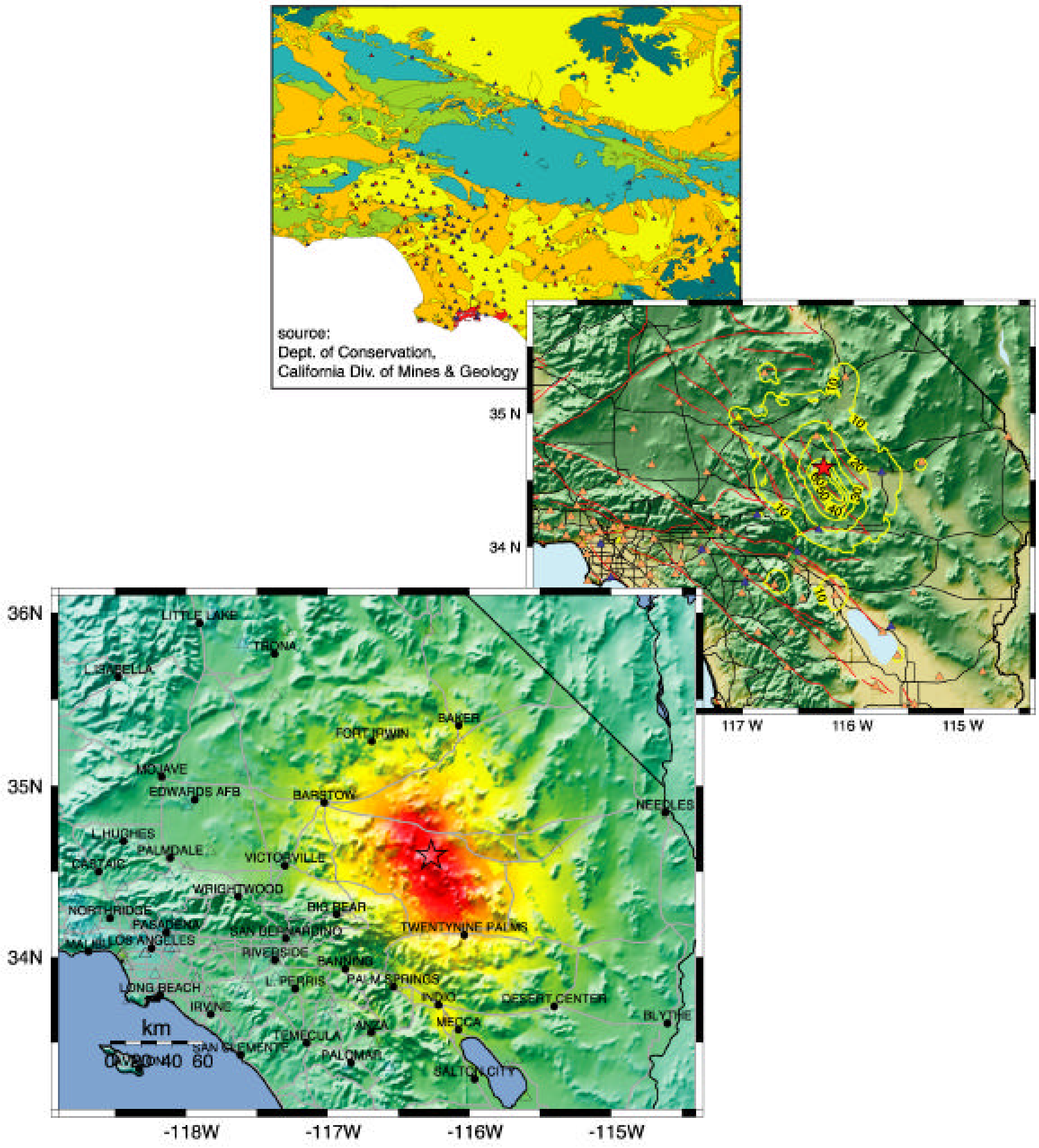


Figure 2. Making a ShakeMap. The example is based on the TriNet ShakeMap for the 1999 Hector Mine, California, earthquake. Triangles show station locations; the epicenter is show with a star, and contours show peak ground acceleration in percent "g". Red lines depict faults; black lines indicate highways. See text for details.

Note that while the near-source ground motions relied on the ShakeMap system's built in predictive tools, well populated areas had good station coverage, so the observed level of shaking there was well-established and available within 4 minutes of the earthquake. That is, while the predictive component of the map improved with time as additional information became available about the earthquake source, shaking reported for the urban areas was well constrained and did not change from the initial maps. The third panel shows the estimate of seismic intensity throughout the region based on the relations established for computing instrumental intensity from peak ground acceleration and velocity as provided by Wald et al. (1999a).

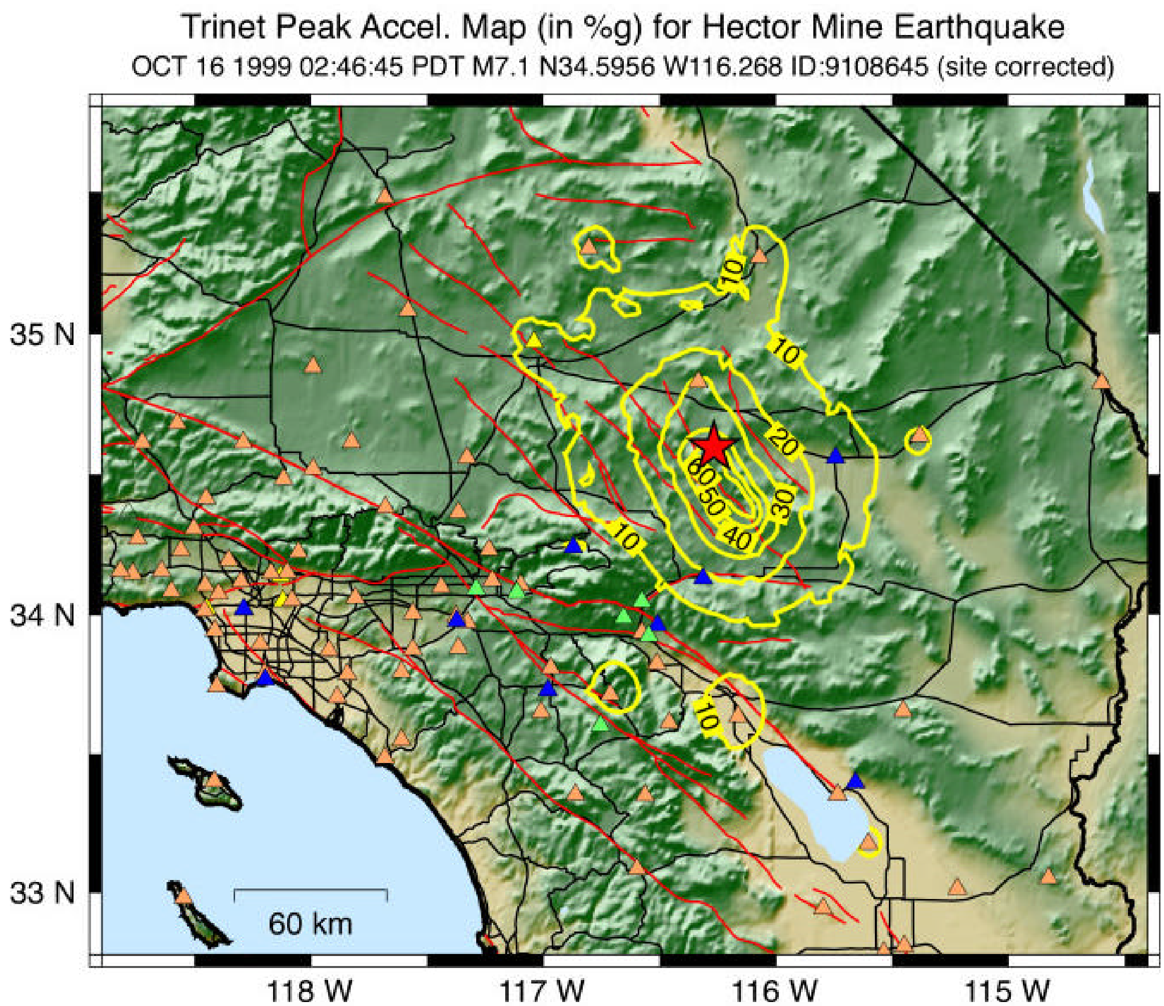
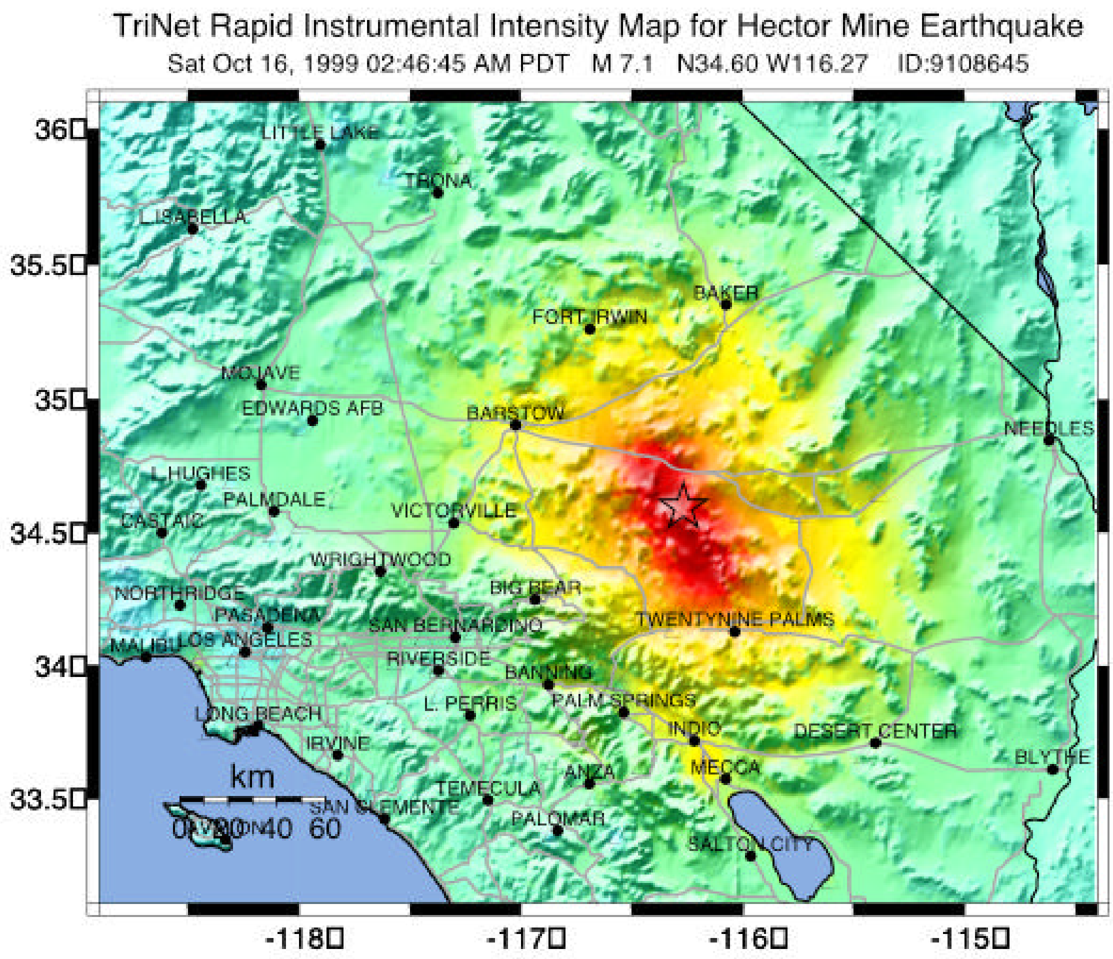


Figure 3. Peak ground acceleration ShakeMap for the 1999 Hector Mine, California Earthquake. Triangles are real-time USGS/Caltech (blue), dial-up CGS (orange), and USGS National Strong Motion Instrument Programs (NSMP) stations (green). Contours of acceleration are given in percent "g". The epicenter is depicted with a red star, mapped faults are shown in red, and the major roadways are given in black. Note there are few stations in the epicentral region.

ShakeMap Examples

In this section we highlight ShakeMaps made for significant earthquakes in each of the past three years. These and other examples are best viewed interactively online on the ShakeMap Web pages (http://www.shakemap.org). Links found on the ShakeMap Web pages contain an archive of all ShakeMaps made to date as well as for major events that occurred prior to the advent of TriNet. These earlier events, e.g., the 1994 Northridge earthquake, were produced with the existing analog data recorded then but processed using the current ShakeMap tools and methodology.



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
PEAK VEL.(cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	8.1-16	16-31	31-60	60-116	>116
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Figure 4. Instrumental Intensity ShakeMap for the October 16, 1999 magnitude 7.1 Hector Mine, California Earthquake.

1999 Hector Mine, California Earthquake

ShakeMaps have been generated in southern California since March 1997. The largest event to be recorded by the new TriNet system and mapped using ShakeMap was the October 16, 1999, magnitude 7.1 Hector Mine earthquake (see Fig. 4). Fortunately, the earthquake occurred in a remote area of the Mojave Desert, so little damage and few injuries were reported. Nevertheless, it was a good opportunity to evaluate the network and test the timeliness and quality of its products. Since the event occurred in a sparsely populated region, the spacing of seismic stations in the near-fault region was also sparse.

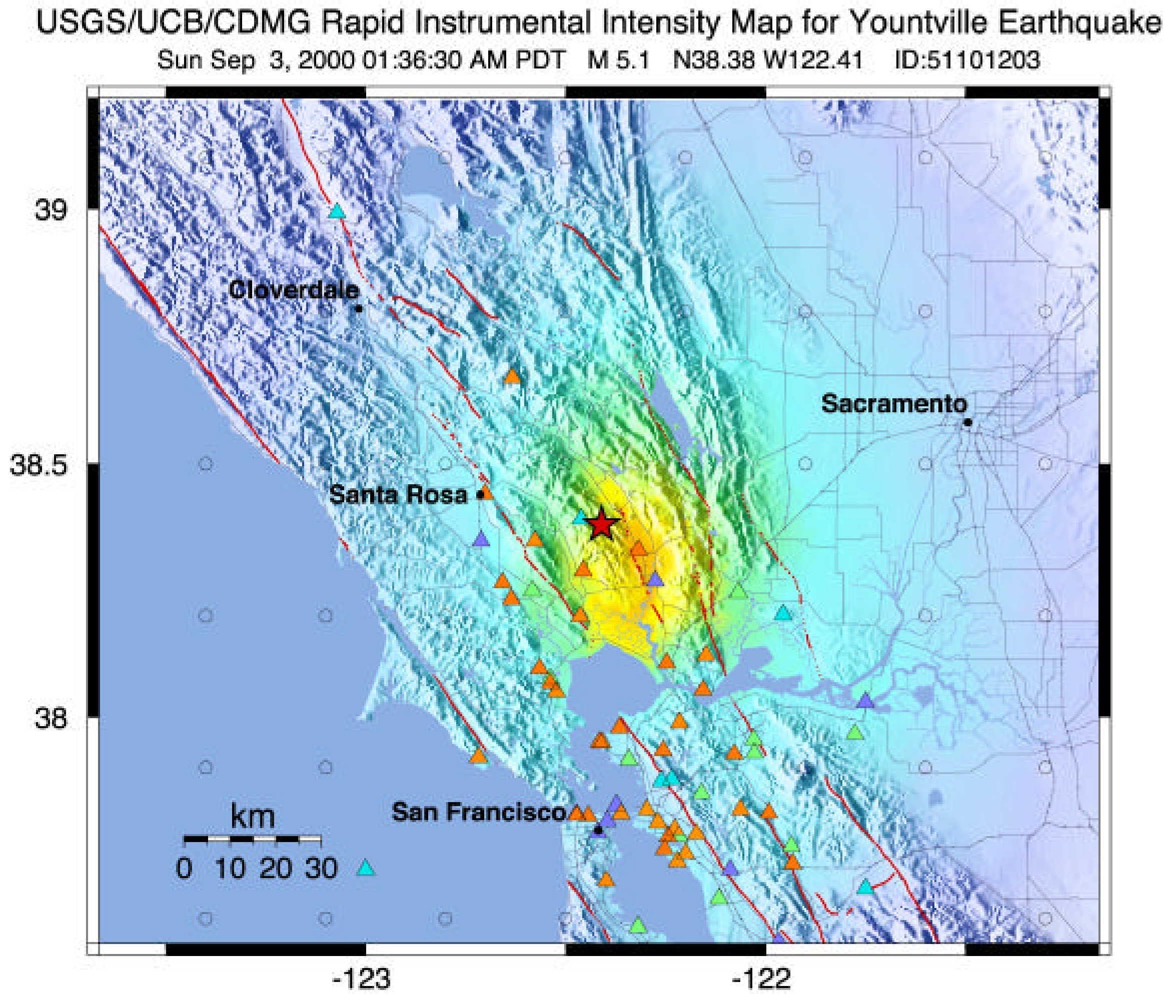
The performance of ShakeMap could be assessed under conditions that might prevail in a more urban earthquake for which near-fault stations might not immediately report due to power or communications failures. The TriNet real-time system determined a magnitude (energy magnitude) of 7.0 within 1 minute of the event, and ShakeMap was successfully produced and distributed within 4 minutes. The ground motion from the Hector Mine event was widely felt in urban Los Angeles and, based on past experience, responders, the media and public had legitimate concerns regarding its source and potential damage. The ShakeMap provided rapid evidence that large-scale emergency response mobilization was unnecessary. The ShakeMap also highlighted areas of amplified ground motion in the Coachella Valley and focused attention on numerous triggered events under the Salton Sea that were within 2 km of the San Andreas fault.

2000 Napa Valley (Yountville), California Earthquake

While moderate in size at magnitude 5.1, the September 3, 2000 Yountville earthquake caused significant damage in the city of Napa. The event occurred in the mountains 6 miles northwest of the city of Napa, near Yountville, California. As shown in Figure 5, the strongest shaking recorded was just north of the city of Napa. The recorded acceleration there was 50 percent of the force of gravity, rather high for this magnitude, but consistent with the significant damage that the city suffered.

Although earthquake shaking levels depend predominantly on the distance from the earthquake source, the high level of ground shaking in Napa appears to have been controlled by two other factors: first, the amplification of shaking by young sediments along the Napa River which shows as a topographic low on the ShakeMap intensity Map (Figure 5) and second, the focusing of strong motion to the southeast, the direction the earthquake rupture appears to have propagated. The offset of the strongest shaking to the southeast from the epicenter, and the amplification within the basin of sediments underlying Napa and along the northern shore of San Pablo Bay are also clear on the map of instrumental intensity.

ShakeMap quality strong motion instrumentation coverage in the San Francisco Bay area has also substantially improved since the 2000 Napa earthquake, so future earthquakes will have better station control.



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
PEAK VEL.(cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	8.1-16	16-31	31-60	60-116	>116
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Figure 5. Instrumental Intensity ShakeMap for the magnitude 5.1 Napa Valley ("Yountville") earthquake on September 3, 2000.

2001 Seattle (Nisqually), Washington Earthquake

Figure 6 shows an example of a ShakeMap for the largest event to date to occur in a region of the country outside of California making ShakeMaps. While the 2001 Nisqually, Washington earthquake was of comparable magnitude to the 1994 Northridge earthquake, the depth of the rupture was much greater—near 50 km. In contrast, the Northridge earthquake rupture was as shallow as 5 km. Primarily as a result of this greater depth, the Nisqually earthquake caused approximately $1/2 billion of damage compared to $40 billion in losses due to the Northridge earthquake.

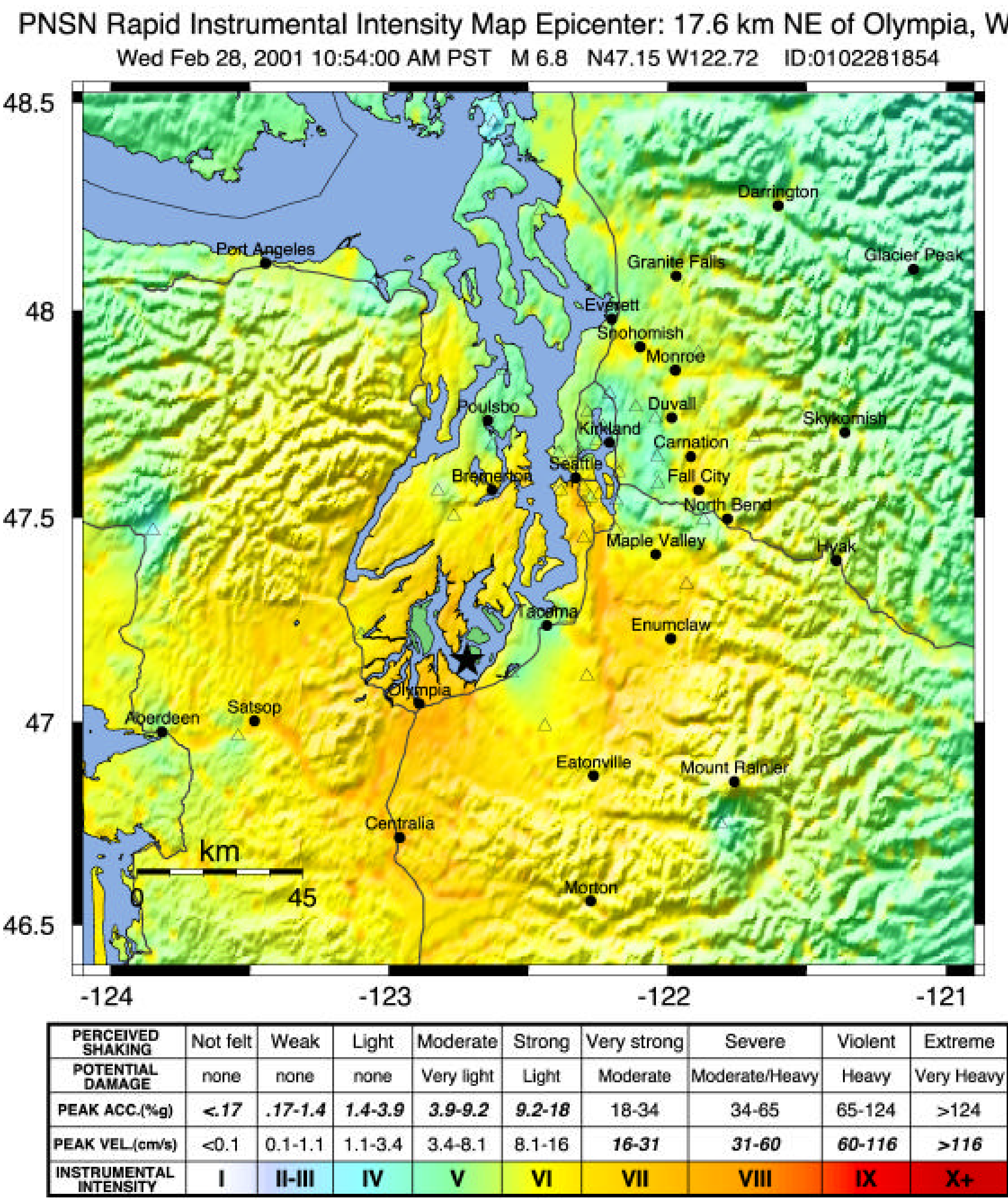


Figure 6. Example ShakeMap in the Pacific Northwest ANSS Region for the 2001 Nisqually, Washington (M6.8) earthquake. Open triangles depict station locations. Note correspondence of intensity of shaking and basin and lowland areas as revealed by the topographic basemap.

The Nisqually earthquake occurred shortly after a major upgrade to the seismic network in the ANSS Pacific Northwest region, and the ShakeMap system in the Seattle region was installed but not fully operable at the time of the quake. Nonetheless, with substantial efforts, ShakeMaps were made available within a day of the event. The

ShakeMap in Figure 6 highlights the utility of comparing shaking intensity atop topographic relief. Since the topography serves as a proxy for site conditions (basins are typically flat, low-lying areas and steep mountains typically are rock), areas of amplified shaking usually correlate well with areas of low relief.

Current Applications of ShakeMap

ShakeMap originated primarily as an Internet-based system for real-time display (see http://www.shakemap.org). While the maps on the Web site are the most visible result of ShakeMap system, they are just one representation of the ShakeMap output. ShakeMap produces grids of acceleration and velocity amplitudes, spectral response values, instrumental intensities, GIS files and a host of other products for designed specific users. In this section, we describe these other products and show how they are being distributed and utilized by different user groups. A summary of the products available on the ShakeMap Web pages for each earthquake is shown in Figure 7.

Emergency Response

The distribution of shaking in a large earthquake, whether expressed as peak acceleration or intensity, provides responding organizations a significant increment of information beyond magnitude and epicenter. Real-time ground shaking maps provide an immediate opportunity to assess the scope of an event, that is, to determine what areas were subject to the highest intensities and probable impacts as well as those which received only weak motions and are likely to be undamaged. These maps will certainly find additional utility in supporting decision-making regarding mobilization of resources, mutual aid, damage assessment, and aid to victims

For example, the Hector Mine earthquake of October 16, 1999 provides an important lesson in the use of ShakeMap to assess the scope of the event and determine the level of mobilization necessary. This earthquake produced ground motion that was widely felt in the Los Angeles basin and, at least in the immediate aftermath, required an assessment of potential impacts. It was rapidly apparent, based on ShakeMap, that the Hector Mine earthquake was not a disaster and despite an extensive area of strong ground shaking, only a few small desert settlements were affected. Thus, mobilization of a response effort was limited to a small number of companies with infrastructure in the region and brief activations of emergency operations centers in San Bernardino and Riverside Counties and the California Office of Emergency Services, Southern Region.

Had a magnitude 7 earthquake occurred in urban Los Angeles or another urban area in California, ShakeMap could have been employed to quickly identify the communities and jurisdictions requiring immediate response. A ShakeMap driven calculation of estimated regional losses would provide focus to the mobilization of resources and expedite the local, state, and federal disaster declaration process, thus initiating the response and recovery machinery of government. ShakeMap, when overlaid on maps featuring critical facilities (e.g. hospitals, police and fire stations, etc.), highways and bridges and vulnerable structures, provides an important means of prioritizing response. Such response activities include: shelter and mass care, search and rescue, medical emergency services, damage and safety assessment, utility and lifeline restoration and emergency public information.

Figure 7. Example of the ShakeMap *Download* web page. A separate page like this is available for each earthquake.

To help facilitate the use of ShakeMap in emergency response, ShakeMap is now provided to organizations with critical emergency response functions automatically through the Internet with "push" technology. These organizations and utilities include the State of California OES, the Los Angeles County Office of Emergency Management, Southern California Edison and the Metropolitan Water District. ShakeMap ground motion maps are now also customized and formatted into Geographic Information Systems (GIS) shapefiles for direct input into the FEMA's U.S. (HAZUS) loss estimation software. These maps are rapidly and automatically distributed to the California Office of Emergency Services (OES) for computing HAZUS loss estimates and for coordinating

State and Federal response efforts. This is a major improvement in loss estimation accuracy since actual ground motion observations are used directly to assess damage rather than relying on simpler estimates based on epicenter and magnitude alone as was customary.

In addition to GIS formatted maps specifically design for HAZUS, we also make shapefiles for more general GIS use. These layers are fundamental as base maps upon which one can overlay a user's infrastructure or inventory. For example, ShakeMaps are also being distributed to regional and state utility providers to enable them to determine areas of their networks that may have sustained damage. Using GIS systems, quick analyses of the situation is possible, and decision-making is greatly facilitated. These GIS maps for both recent and past earthquakes are now routinely used by Insurance, Engineering, Financial institutions, and others.

Public Information and Education

The rapid availability of ShakeMap on the Internet combined with the urgent desire for information following a significant earthquake makes this mapping tool a compelling source of emergency public information and education. In instances in which an earthquake receives significant news coverage, the ShakeMap site as well as the Community Internet Intensity Map[1] (which poses the question, "Did you feel it?") may receive an enormous increase in website visitors.

On October 16, 1999, local television stations devoted considerable airtime to the Hector Mine earthquake. During live news briefings, Caltech and USGS scientists employed ShakeMap to discuss the event, invited viewers to visit the ShakeMap website and posted the web address prominently above the podium in the Media Center. By the end of the day, the ShakeMap website had received over 300,000 visitors. Even for small events rapid and reliable earthquake information is important. For instance, on January 13, 2001, when two magnitude 4 events, centered in the northeast San Fernando Valley area of Los Angeles, were followed by local news coverage, web visits peaked at 233 hits per *second*.

Acknowledging the importance of ShakeMap as a tool for public information and education, we developed a "TV" ShakeMap in cooperation with regional news organizations. This version of ShakeMap represents a substantial simplification of the "official" map that appears on the ShakeMap website. Based on recommendations of news representatives, acceleration and velocity were omitted from the TV version of ShakeMap. Concern that magnitude and intensity might be confused prompted removal of Roman numerals representing intensity and intensity was depicted using only the color bar. Magnitude and location were enlarged and posted at the top of the map (see Figure 10).

[1] Invites web visitors (http://pasadena.wr.usgs.gov/shake) to record their observations on a questionnaire and the data obtained are aggregated to establish a zip code-based intensity profile for the event (See Wald et al., 1999c, for more details).

The ShakeMap for television audiences was developed specifically to encourage broadcast journalists to provide a more accurate depiction of earthquakes in news reports. Prior to ShakeMap, the typical visual representation of an earthquake consisted of a map overlay with the epicenter and radiating concentric rings to represent ground motion. The patterns of ground motion are not symmetrical as suggested by these illustrations and represent an underutilization of available technology by the news media. Use of ShakeMap to discuss an earthquake that has just occurred not only provides a more accurate image of earthquake ground motion patterns, it also provides important additional information regarding the potential severity of shaking that is useful both to residents of the area impacted and those outside the area who are concerned about friends and family.

ShakeMaps are now reaching a much wider audience through television broadcasting than would be possible through the Internet alone. As an example, a recent magnitude 4.2 earthquake near Valencia on January 28, 2002, was felt throughout the San Fernando Valley and northern Los Angeles basin, occurred at 9:54 pm. At least one local news organization *lead* the 10 o'clock News with a ShakeMap image providing information about the distribution of shaking to millions of viewers only six minutes after the shaking.

Earthquake Engineering and Seismological Research

For potentially damaging earthquakes, ShakeMap also produces response spectral values for use not only in loss estimation as mentioned earlier, but also for earthquake engineering analyses. Response spectra for a given location are useful for portraying the potential effects of shaking on particular types of buildings and structures. Following a damaging earthquake, ShakeMaps of spectral response will be key for prioritizing and focusing post-earthquake occupancy and damage inspection by civil engineers.

In addition to providing information on recent events, ShakeMap Web pages provide maps of the shaking and ground motion parameters for past significant earthquakes. Engineers can use these maps to understand the maximum and cumulative effects of seismic loading for the life of any particular structure. This is particularly relevant given the recent discovery of the potential damage to column/beam welds in steel buildings following the 1994 Northridge earthquake.

In seismological research, ShakeMap has been proven particularly effective in gaining a quick overview of the effects of geological structure and earthquake rupture processes on the nature of recorded ground motions. ShakeMaps showing the distribution of recorded peak ground acceleration and velocity overlain on regional topography maps allow scientists to gauge the effects of local site amplification since topography is a simple proxy for rock versus deep basin soil site conditions. This can lead to more detailed investigations into the nature of the controlling factors in generating localized regions of damaging ground motions.

Planning and Training: ShakeMap Earthquake Scenarios

In planning and coordinating emergency response, utilities, local government, and other organizations are best served by conducting training exercises based on realistic earthquake situations—ones that they are most likely to face. Scenario earthquakes can fill this role. The ShakeMap system can be used to map ground motion estimates for earthquake scenarios as well as real data. Scenario maps can be used to examine exposure of structures, lifelines, utilities, and transportation conduits to specific potential earthquakes. ShakeMap automatically includes local effects due to site conditions**.** The ShakeMap Web pages now have a special section under the *Map Archives* pages that display selected earthquake scenarios (www.trinet.org/shake/archive/scenario/html). Additional scenario events will be supplied as they are requested and generated. To contact the ShakeMap Working Group, please use the comment form available on the Web site.

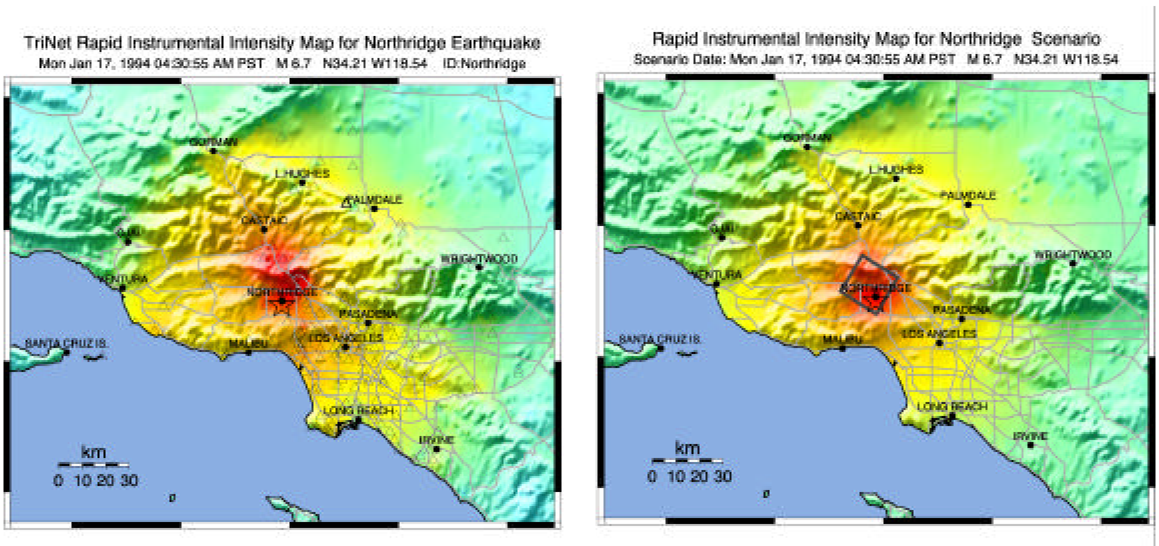


Figure 8. Northridge Earthquake ShakeMap (Left) and scenario earthquake (Right) for the Northridge earthquake made by assuming the correct magnitude and fault rupture area shown projected to the surface (black rectangle).

Given a selected event, we have developed tools to make it relatively easy to generate a ShakeMap earthquake scenario. First we need to assume a particular fault or fault segment will (or did) rupture over a certain length or segment. We then determine the magnitude of the earthquake based on assumed rupture dimensions. Next, we estimate the ground shaking at all locations in the chosen area around the fault, and then represent these motions visually by producing ShakeMaps. The scenario earthquake ground motion maps are identical to those made for real earthquakes---with one exception: ShakeMap scenarios are labeled with the word “SCENARIO” prominently displayed to avoid potential confusion with real earthquake occurrences (see Figure 9).

At present, ground motions are estimated using empirical attenuation relationships to estimate peak ground motions on rock conditions. We then correct the amplitude at that location based on the local site soil (NEHRP) conditions as we do in the general

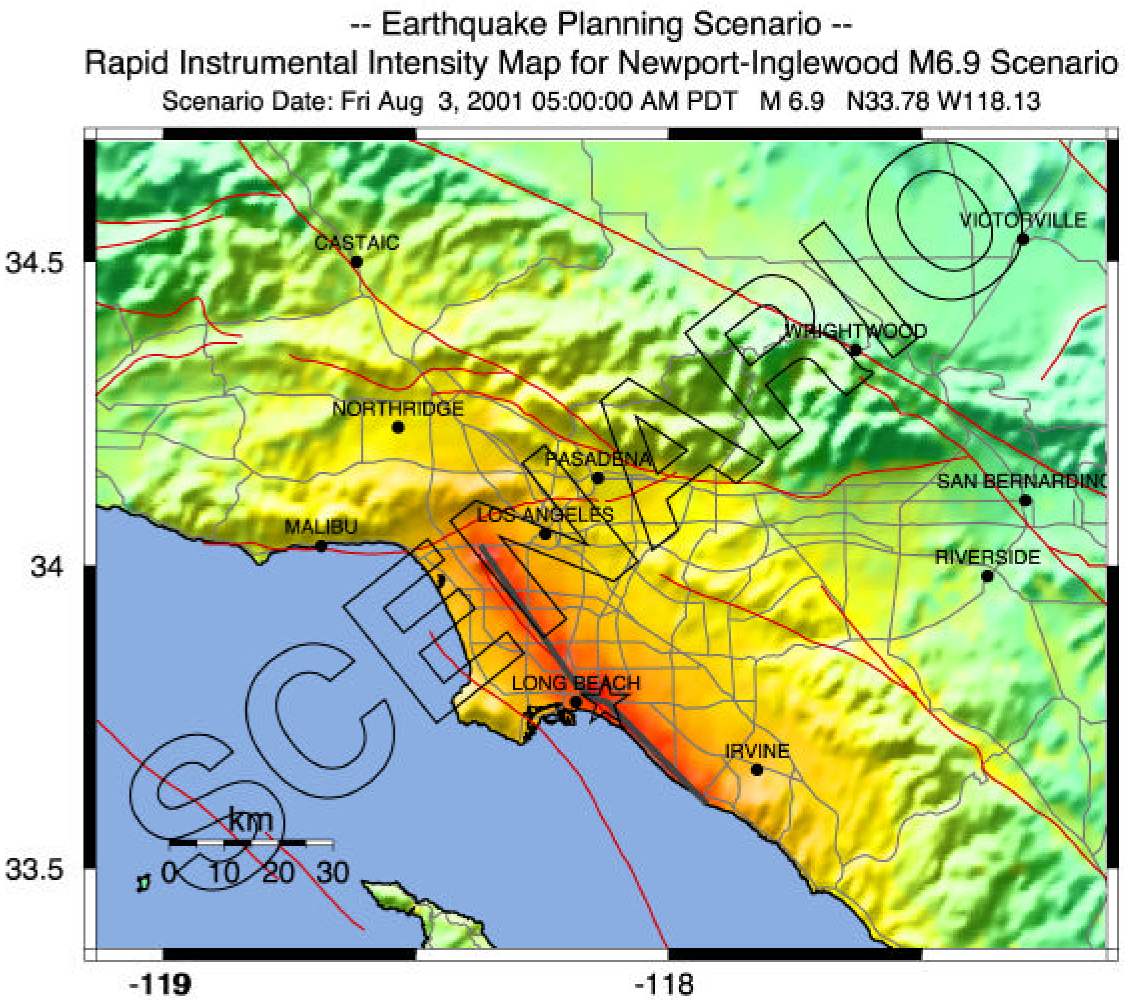
ShakeMap interpolation scheme. Finiteness is included explicitly, but directivity enters only through the empirical relations. As an example of the effectiveness of the scenario generation process, Figure 8 shows both the observed ShakeMap for the 1994 Northridge earthquake (left) and a estimated ShakeMap scenario (right) computed with the same earthquake source information assumed in the typical scenario calculations: the magnitude and geometry of the fault that slipped. In this case the dimensions of the Northridge rupture are known from analyses of the earthquake source (e.g., Wald et al., 1996).

In the current ShakeMap scenarios, we do not explicitly include the effects of rupture directivity, which has been shown to concentrate energy and the strongest shaking away from the hypocenter and in the direction that the fault rupture progresses. In Figure 8, the observed shaking from the Northridge earthquake (left) has more energy in the region northwest of the epicenter than the scenario version (Figure 8, right). This is due to the fact that the earthquake indeed exhibited northwestward directivity, and ShakeMap includes this only in an average sense in the predictions for the scenario. However, much of the shaking pattern is recovered just by knowing the dimensions of the fault that ruptured. In the case of strike slip earthquakes like the Newport-Inglewood and San Andreas fault (Ft. Tejon) scenarios, directivity can be quite severe, so depending on where the actual epicenter is, the shaking pattern might be skewed toward stronger shaking away from the epicenter than is shown in are scenarios. Likewise, the ground motion estimates we use for the scenarios are median values, and real data will have greater amplitude variations, potentially over small spatial scales.

Our ShakeMap earthquake scenarios are an integral part of emergency response planning. Primary users include city, county, state and federal government agencies (e.g., the California Office of Emergency Services, FEMA, the Army Corp of Engineers) and emergency response planners and managers for utilities, businesses, and other large organizations. Scenarios are particularly useful in planning and exercises when combined with bss estimation systems such as HAZUS and the Early Post-Earthquake Damage Assessment Tool (EPEDAT), which provide scenario-based estimates of social and economic impacts.

Depending on the level of complexity needed for the scenario, event-specific factors such as directivity and variable slip distribution could also be incorporated in the amplitude estimates fed to ShakeMap. Scenarios are of fundamental interest to scientific audiences interested in the nature of the ground shaking likely experienced in past earthquakes as well as the possible effects due to rupture on known faults in the future. In addition, more detailed and careful analysis of the ground motion time histories (seismograms) produced by such scenario earthquakes is highly beneficial for earthquake engineering considerations. Engineers require site-specific ground motions for detailed structural response analysis of existing structures and future structures designed around specified performance levels. In the near future, these scenarios will also provide synthetic time histories of strong ground motions that include rupture directivity effects.

The U.S. Geological Survey has evaluated the probabilistic hazard from active faults in the United States for the National Seismic Hazard Mapping Project. From these maps it is possible to prioritize the best scenario earthquakes to be used in planning exercises by considering the most likely candidate earthquake fault first, followed by the next likely, and so on. Such an analysis is easily accomplished by hazard disaggregation, in which the contributions of individual earthquakes to the total seismic hazard, their probability of occurrence and the severity of the ground motions, are ranked. Using the individual components ("disaggregations") of these hazard maps, a user can properly select the appropriate scenarios given their location, regional extent, and specific planning requirements.



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
PEAK VEL.(cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	8.1-16	16-31	31-60	60-116	>116
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Figure 9. An example Scenario Earthquake ShakeMap based on a hypothetical magnitude 6.9 rupture on the Newport-Inglewood fault in Los Angeles. Note that the

word “Scenario” is featured quite prominently to avoid confusing ShakeMap Scenarios with the real thing.

An example of a ShakeMap scenario earthquake is shown in Figure 9 for a hypothetical magnitude 7.1 earthquake on the Newport-Inglewood fault near Los Angeles. Due to the proximity to populated regions of Los Angeles, this scenario represents one the most destructive earthquakes that could impact the region. The U.S. Army Corp of Engineers recently used an event similar to this scenario for evaluating their capacity to respond to such a disaster and to continue to build cooperative relationships with other Federal, State, and local emergency response partners.

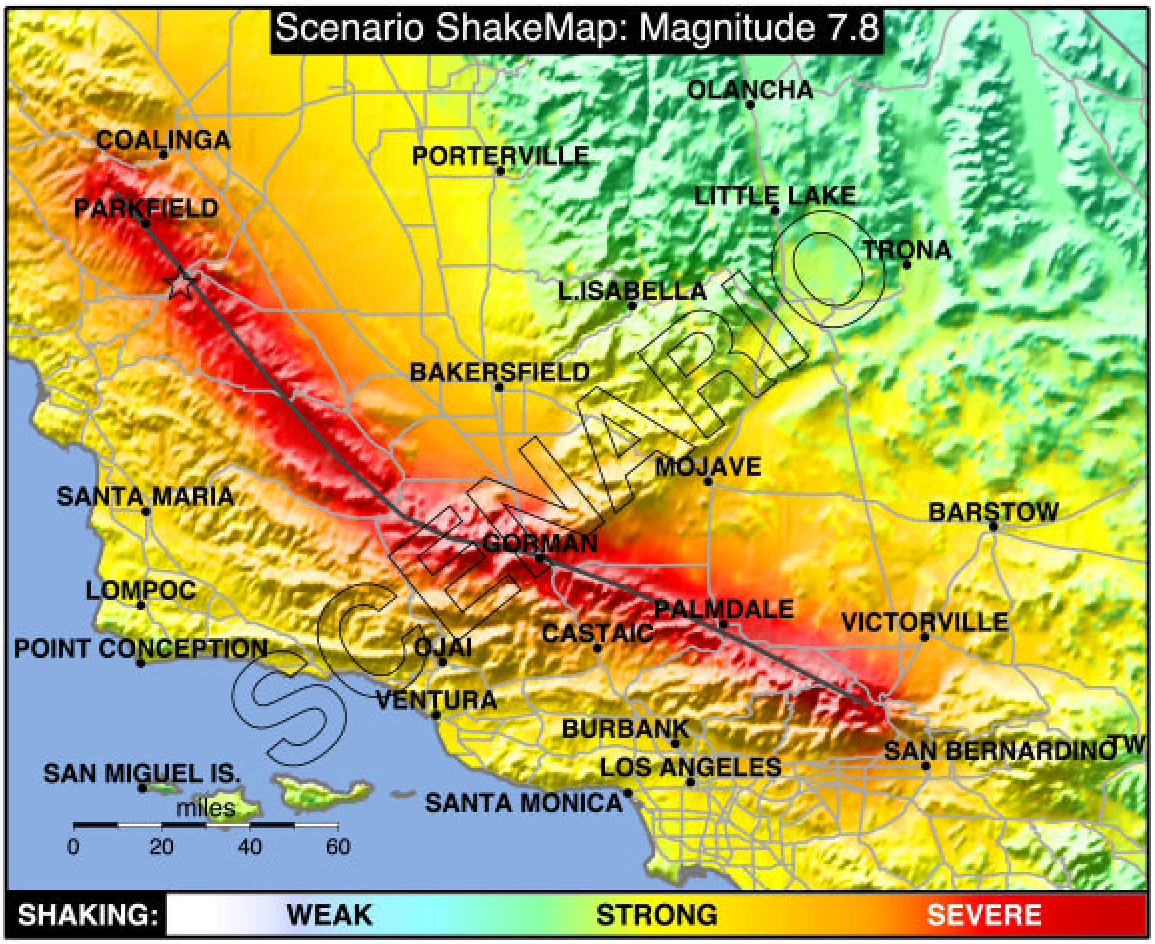
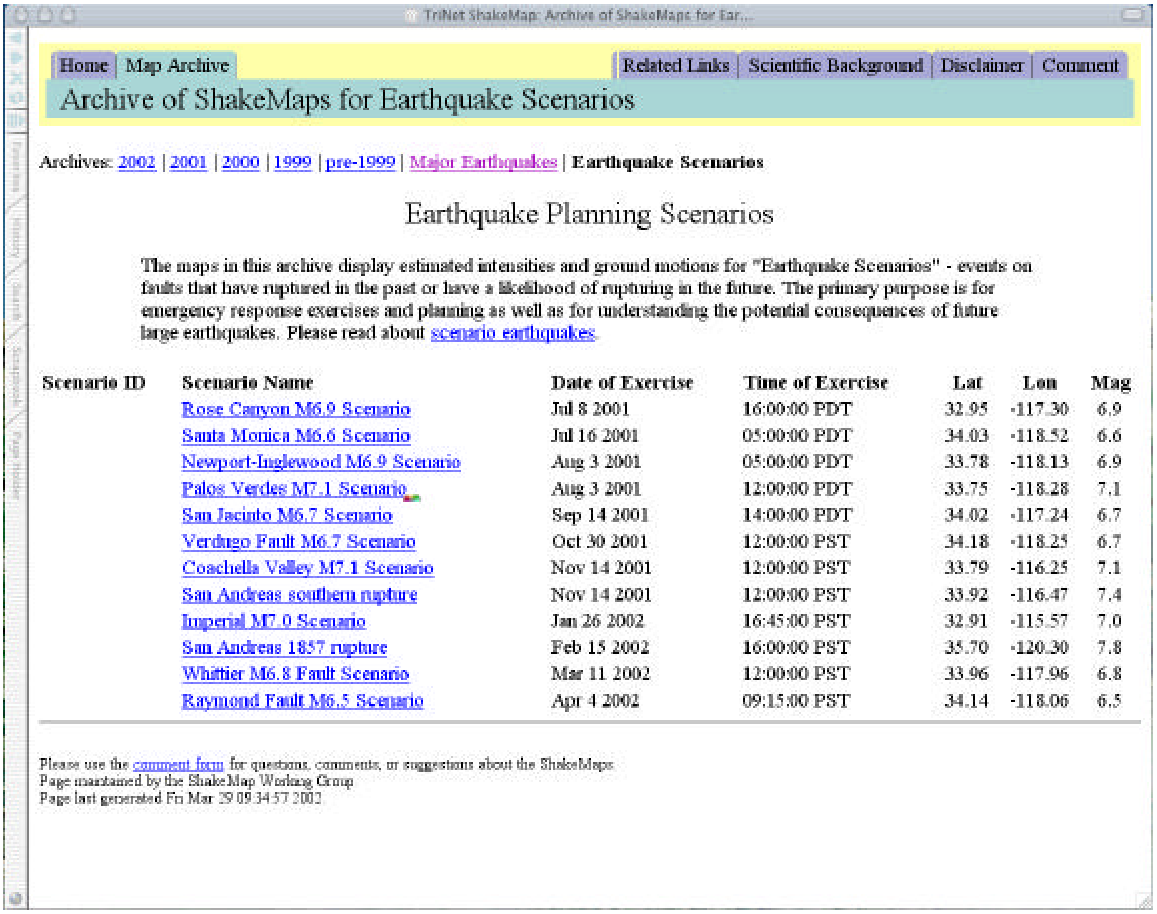


Figure 10. Scenario ShakeMap intensity based on a repeat of the great magnitude 7.8 Fort Tejon, California, earthquake which occurred in 1857. This format of the ShakeMap is the “TV” version, with larger text and features and a simplified legend suitable for television broadcasting.

The next example of a scenario earthquake represents a repeat of the great 1857 Fort Tejon earthquake. The length of the rupture is well established from paleo-seismological studies. This scenario represents a rough estimate of the possible shaking distribution for southern California's "Big One". The scenario, shown in Figure 10, is portrayed in the "TV" ShakeMap format, which simplifies the legend for a more general audience as well as accommodates the lower resolution aspects of TV screens compared to computer monitors.

These and other scenarios are available online at the ShakeMap web pages. They are formatted the same as other ShakeMaps, so they too can be easily used in for response planning and loss estimation as well as for educational purposes. They can be found from the *Map Archive* link at the top of all ShakeMap Web pages.


TriNet ShakeMap: Archive of ShakeMaps for Ear...

Home | Map Archive | Related Links | Scientific Background | Disclaimer | Comment

Archive of ShakeMaps for Earthquake Scenarios

Archives: 2002 | 2001 | 2000 | 1999 | pre-1999 | Major Earthquakes | **Earthquake Scenarios**

Earthquake Planning Scenarios

The maps in this archive display estimated intensities and ground motions for "Earthquake Scenarios" - events on faults that have ruptured in the past or have a likelihood of rupturing in the future. The primary purpose is for emergency response exercises and planning as well as for understanding the potential consequences of future large earthquakes. Please read about scenario earthquakes.

Scenario ID	Scenario Name	Date of Exercise	Time of Exercise	Lat	Lon	Mag
	Rose Canyon M6.9 Scenario	Jul 8 2001	16:00:00 PDT	32.95	-117.30	6.9
	Santa Monica M6.6 Scenario	Jul 16 2001	05:00:00 PDT	34.03	-118.52	6.6
	Newport-Inglewood M6.9 Scenario	Aug 3 2001	05:00:00 PDT	33.78	-118.13	6.9
	Palos Verdes M7.1 Scenario	Aug 3 2001	12:00:00 PDT	33.75	-118.28	7.1
	San Jacinto M6.7 Scenario	Sep 14 2001	14:00:00 PDT	34.02	-117.24	6.7
	Verdugo Fault M6.7 Scenario	Oct 30 2001	12:00:00 PST	34.18	-118.25	6.7
	Coachella Valley M7.1 Scenario	Nov 14 2001	12:00:00 PST	33.79	-116.25	7.1
	San Andreas southern rupture	Nov 14 2001	12:00:00 PST	33.92	-116.47	7.4
	Imperial M7.0 Scenario	Jan 26 2002	16:45:00 PST	32.91	-115.57	7.0
	San Andreas 1857 rupture	Feb 15 2002	16:00:00 PST	35.70	-120.30	7.8
	Whittier M6.8 Fault Scenario	Mar 11 2002	12:00:00 PST	33.96	-117.96	6.8
	Raymond Fault M6.5 Scenario	Apr 4 2002	09:15:00 PST	34.14	-118.06	6.5

Please use the comment form for questions, comments, or suggestions about the ShakeMaps.
Page maintained by the ShakeMap Working Group
Page last generated Fri Mar 29 09:34:57 2002

Figure 11. Scenario ShakeMap Web pages for southern California. Events are added upon request (see *Comment* link on Web pages).

Future Developments

Ongoing development involves automatically generated, interactive GIS applications for ShakeMap users who are either familiar with or who have expertise in GIS tools and applications. We are implementing both server-side and client-side applications to ensure both diversity of GIS tools and robust access during the immediate post earthquake time period. Server-side tools allow fully interactive overlays of a variety of ShakeMap parameters and maps with a wide range of regional infrastructure, but their availability cannot be guaranteed in the minutes immediately following a damaging earthquake due extreme demands on the server. In contrast, client-side GIS applications are less versatile, but can be made robust by rapidly and automatically delivering the ShakeMap GIS content (shapefiles) to users. These interfaces will be available in the near future.

ShakeMap software has been developed for reliable and robust operation. In addition, the software architecture was designed to be directly portable to other regions of the country. Operating ShakeMap systems now in place cover California as well as the Seattle and Salt Lake City areas. As more seismometers are installed under the Advanced National Seismic System, ShakeMap coverage will be expanded. Regions that will likely come online in the near future include the environs of Memphis Tennessee, Anchorage Alaska, Reno Nevada, and Puerto Rico.

Conclusions

ShakeMap is a powerful tool that provides a detailed, graphical summary of ground shaking due to earthquakes. Following a major earthquake, it can be put to use by emergency responders to identify the areas most likely to have suffered heavy damage. Efforts are being made to add dedicated links to more government agencies and utilities providers to ensure that the data are received and to provide data in formats that enable them to utilize the information in existing response systems. In connection with probabilistic hazard maps, ShakeMaps based on earthquake scenarios can also be used to identify points of exposure in lifelines and major structures and to evaluate emergency response plans. By producing a wide range of products and maps, ShakeMap is also of value to earthquake engineers and earth scientists, as well as the general public through the Internet and the News media.

Acknowledgements

ShakeMap is one important end product of a very sophisticated seismic network. It can only be produced within the context of a robust, real-time seismic operation. It is to the credit of all involved with the networks including those within the CGS, Caltech, USGS, U.C. Berkeley, and other contributors.

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GUIDELINES FOR UTILIZING STRONG-MOTION AND SHAKEMAP DATA IN POST-EARTHQUAKE RESPONSE: AN OVERVIEW

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Abstract

The ATC-54 Report, *Guidelines for Using Strong-Motion Data for Postearthquake Response and Postearthquake Structural Evaluation*, under preparation by the Applied Technology Council for the California Division of Mines and Geology, provides guidance on (1) the use of near-real-time computer-generated ground-motion maps in emergency response, and (2) the use and interpretation of strong-motion data to evaluate the earthquake response of buildings, bridges, and dams in the immediate postearthquake aftermath. Guidance is also provided on the collection of data describing the characteristics and performance of structures in which, or near which, strong-motion data have been recorded.

Introduction

Background

Since the installation of the initial network of nine strong-motion instruments at ground sites and in buildings in California in 1932 (Matthiesen, 1980), the number of strong-motion recording stations and records has grown dramatically. Today there are more than 1000 instrumented sites and structures in California, including buildings, dams, bridges, and other lifeline structures. The instruments are operated by a wide variety of agencies and owners, including the California Division of Mines and Geology (CDMG), California Division of Water Resources, California Department of Transportation, U. S. Geological Survey (USGS), U.S. Bureau of Reclamation, U.S. Army Corp of Engineers, several universities and university-affiliated centers, utility companies in northern and southern California, and owners of buildings where instruments have been mandated by building code requirements. Hundreds of strong-motion time histories have been recorded at these stations, resulting primarily from large damaging earthquakes, such as the 1971 San Fernando, 1989 Loma Prieta, and 1994 Northridge earthquakes. Such data are available in digital form from the principal network operators (CDMG and the USGS) and other sources, including the world wide web virtual data center operated by the Consortium of Organizations for Strong-Motion Observation Systems (COSMOS).

Over the last 40 to 50 years, the technology for recording, analyzing, and representing strong-motion data has also advanced significantly. Major advances have included: the development of rapid scanning and processing techniques for converting photographic analog records to digital format; the development and deployment of digital accelerographs; the development of new computer analytical methods that use strong-motion records to verify and refine computer models of structural response and to compute estimated component forces and displacements; and, most recently, the introduction of computer-generated ground-motion maps that provide overviews of the regional distribution of ground shaking within minutes, and without human intervention, after damaging earthquakes.

Collectively the existing network of strong-motion instruments, the existing sets of strong-motion data, and the available techniques and technology for processing, analyzing, and displaying strong-motion data provide an ideal set of tools and information for postearthquake response planning and execution, as well as postearthquake evaluation of structures. In recognition of the enormous potential of these tools and information, and with the realization that practicing professionals do not have guidance readily available on how to take advantage of these current technical capabilities, CDMG awarded a Year 2000 California Strong-Motion Instrumentation Program (CSMIP) Data Interpretation Project to the Applied Technology Council (ATC) to prepare the needed guidance. Specifically, the contract required that ATC develop *Guidelines* to: (1) facilitate improved emergency response with the use of near-real-time computer-generated ground-motion maps and (2) facilitate postearthquake evaluation of structures using strong-motion data from ground sites and instrumented buildings, bridges, and dams. Under this project ATC will also provide guidance on the collection of data describing the characteristics and performance of structures in which, or near which, strong-motion data have been recorded.

Guidelines Development Process

Now under development by ATC, the *Guidelines* are being developed through a multi-step approach by a multi-disciplinary team of experienced specialists in earthquake and geotechnical engineering, risk analysis, geographic information systems (GIS), and emergency response planning. Initially, the project team identified and described the state-of-the-art in available data resources, building and lifeline inventory data, GIS hazard maps, and loss estimation tools. The next step was to define the state-of-the-practice in emergency response planning at the state, regional, and local level, as well as in postearthquake structural surveys and evaluations. Based on this information, primarily developed through literature reviews and interviews with key individuals in various agencies and organizations throughout the state, an assessment was made of the existing capabilities in emergency response planning and postearthquake evaluation of structures. This assessment served as the basis for determining the level of information and extent of guidance to be provided in the *Guidelines*. Upon completion, the *Guidelines* will be presented in draft form at a Users' Workshop organized to solicit input on the draft. The final version of the *Guidelines* will be based on input received at the Users' Workshop, as well as review comments from the CSMIP staff and the California Seismic Safety Commission's Strong-Motion Instrumentation Advisory Committee (SMIAC).

Paper Focus and Contents

This paper is one of three papers presented in the SMIP01 Seminar describing the contents of the resource document being prepared under this project, namely *Guidelines for Using Strong-Motion Data for Postearthquake Response and Postearthquake Structural Evaluation*, to be published as the ATC-54 Report (ATC, in preparation). The intent of this paper is to provide an overview of the ATC-54 *Guidelines* and pertinent background information, including a description of the format, content, and preparation of computer-generated ground-motion maps, a new technology that shows promise for emergency response planning and execution. We begin with a description of the purpose and scope of the *Guidelines*, followed by a brief description of computer-generated ground-motion maps. The remainder of the paper is devoted to a description of the *Guidelines* contents and to a set of preliminary recommendations for improving the use of strong-motion data and maps in postearthquake response planning and execution and postearthquake evaluation of structures. The companion papers, "Guidelines for Utilizing ShakeMaps for Emergency Response", by S.A. King et al., and "Guidelines for Utilizing Strong-Motion Data for Evaluation of Structures", by A. G. Brady and C. Rojahn, provide more in-depth information pertaining to the use of ShakeMaps and to the use of strong-motion data for structural evaluation, respectively.

Purpose And Scope Of The *Guidelines*

The *Guidelines* are intended to increase the utilization of strong ground motion data for improving postearthquake response and postearthquake evaluation of buildings, bridges, and dams. They are also intended, as is the goal of all CSMIP data utilization projects, to improve the understanding of strong ground shaking and the response of structures so as to improve seismic design codes and practices. This document is not intended to be a loss-estimation methodology; however, as discussed in the *Guidelines*, much emphasis is placed on the use of strong-motion data within existing loss-estimation tools for estimating the regional impacts of earthquakes.

The audience for this document is diverse and includes local, regional, and state agencies with postearthquake responsibilities; design professionals; facility owners; policy makers; and researchers concerned with the various uses of strong ground-motion data. It is anticipated that most readers will not be interested in all sections of the *Guidelines*.

The *Guidelines* focus on two distinct topics. The first concerns effective means for using computer-generated ground-motion maps in postearthquake response. The intended use of this part of the document is to provide guidance on the development and implementation of applications using such maps for emergency response. Specifically, the applications focus on assessing the following:

- extent of damaged buildings and planning related safety evaluation inspections
- condition of hospitals and other emergency response structures
- impact on utility systems and transportation networks
- extent of liquefaction, landslide, and inundation
- casualties and associated need for victim extraction from damaged structures
- extent of debris from collapsed structures
- sheltering needs

- extent of possible hazardous materials release
- insurance claims
- other postearthquake disaster and recovery ramifications

With respect to these applications, the *Guidelines* are intended to help users evaluate existing practices and policies, plan for future improvements, coordinate mutual aid, allocate resources, and design and budget for mitigation and planning exercises and programs.

The second topic concerns the rapid utilization of near-real-time instrumental recordings from ground and structure stations for postearthquake response and evaluation of structures. In this regard, the *Guidelines* are intended to help with damage determination, rapid estimation of structural distortions (e.g., inter-story displacements), and mathematical model identification and verification. Information is provided on (1) how to interpret data from strong-motion instruments to evaluate structural response rapidly, and (2) the form, type, and extent of data (in the immediate earthquake aftermath) to be collected from structures in the vicinity of strong-motion recordings.

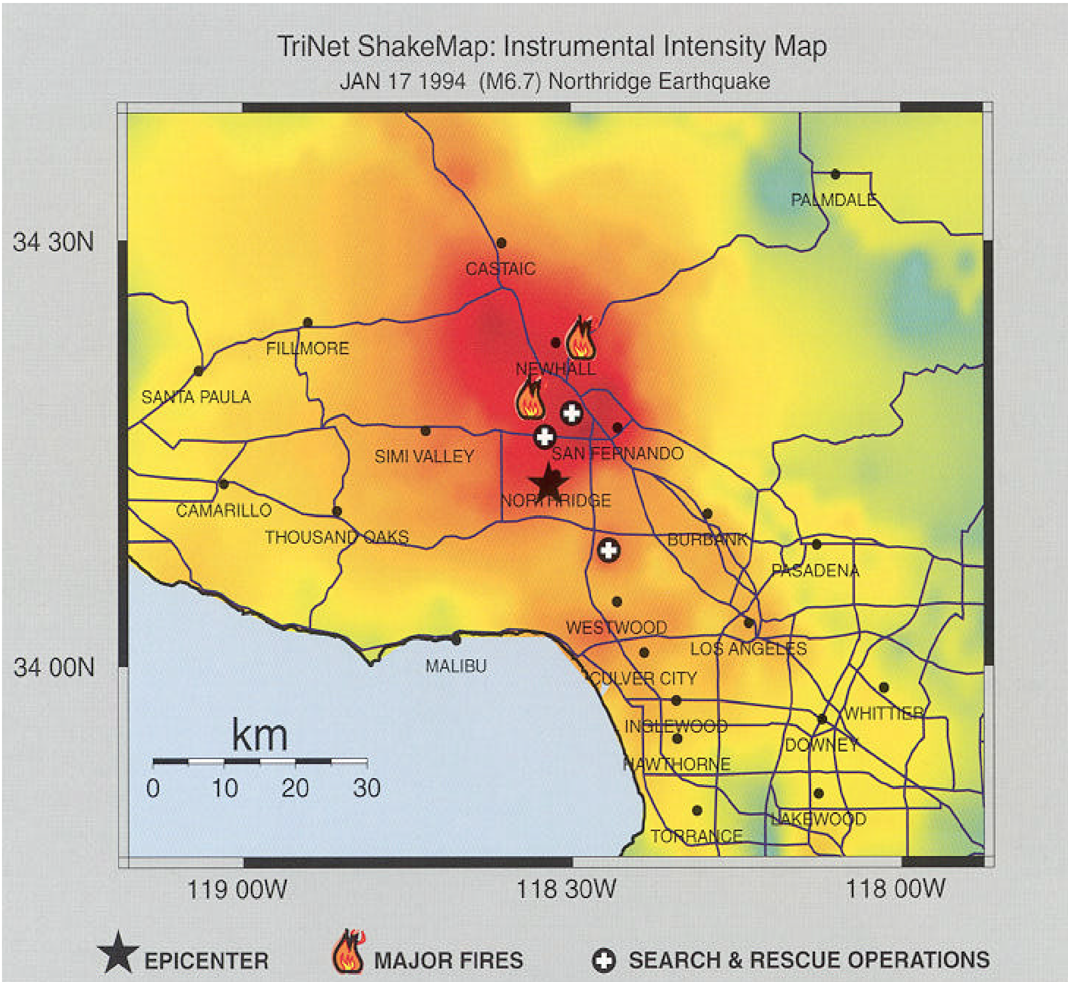
No new research, other than the determination of the state-of-the-art and state-of-the-practice, was undertaken under this project; rather the intent was to create one resource document containing broad guidance on the use of ShakeMaps and currently available resources and techniques for rapid evaluation of structures using strong-motion data.

Computer-Generated Ground-Motion Maps

One of the primary focuses of the *Guidelines* is on the computer-generated ground-motion maps produced by the TriNet program. TriNet is a five-year collaborative effort among the California Institute of Technology (Caltech), the U. S. Geological Survey, and the California Division of Mines and Geology to create an effective real-time earthquake information system for southern California and eventually northern California. A complete description of the history, background, and products of TriNet is available on the web site www.trinet.org.

TriNet ShakeMaps, an example of which is shown in Figure 1, are representations of the ground shaking produced by earthquakes. They are generated automatically following moderate and large earthquakes. These are preliminary ground shaking maps, normally posted within several minutes of the earthquake origin time. They show the distribution of peak ground acceleration and velocity, spectral acceleration at three periods, and an instrumentally-derived, estimated distribution of Modified Mercalli Intensity. The Instrumental Intensity Map is based on a combined regression of recorded peak acceleration and velocity amplitudes. In order to stabilize contouring and minimize the misrepresentation of the ground-motion pattern due to data gaps, the data are augmented with predicted values in areas without recorded data. Given the epicenter and magnitude, peak motion amplitudes in sparse regions are estimated from attenuation curves. As the real-time TriNet station density increases with the passage of time, the reliance on predicted values will decrease.

In addition to producing near-real-time ground-shaking maps, the TriNet ShakeMap program also produces earthquake scenario ground-shaking maps. The earthquake scenarios describe the expected ground motions and effects of specific hypothetical large earthquakes. The maps are used in planning and coordinating emergency response by utilities, emergency responders, and other agencies. The scenario earthquakes provide a more realistic example for training exercises and loss-estimation studies, and can be generated for any hypothetical or historic earthquake.



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
PEAK VEL. (cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	8.1-16	16-31	31-60	60-116	>116
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Figure 1. TriNet ShakeMap for the 1994 Northridge, California earthquake (USGS, 2000).

The steps involve assuming that a particular fault or fault segment will (or did) rupture over a certain length, estimating the likely magnitude of the earthquake, and estimating the ground shaking at all locations in the chosen area around the fault. The ground motions are estimated using an empirical attenuation relationship, which is a predictive relationship that allows the estimation of the peak ground motions at a given distance and for an assumed magnitude.

The web address for the TriNet ShakeMaps is www.trinet.org/shake/. Users of the *Guidelines* are encouraged to visit this site often, not only for the near-real-time ground shaking maps, but also for the new or improved products that are periodically added to the web site.

Organization of the *Guidelines*

The *Guidelines* are intended to be used by a diverse audience, many of whom will only be interested in specific sections of the document. In addition, the document is written for the most basic level of user, so more advanced users will likely be able to skim certain sections within their areas of interest.

The *Guidelines* are organized into five chapters so that users will be able to target quickly their sections of interest (Figure 2). Chapter 1 contains introductory material and pertinent background information. Chapters 2 through 4 provide procedures for using strong ground-motion maps and recordings in emergency response, for evaluating the performance of individual buildings, bridges and dams, and for collecting and documenting postearthquake investigation data, respectively. The final chapter provides a summary of the *Guidelines* and highlights recommendations for more effectively utilizing strong ground-motion maps and data.

ATC-54: *Guidelines for Using Strong-Motion Data for Postearthquake Response and Postearthquake Structural Evaluation*

Contents

Figure 2. Guidelines Table of Contents

Chapter 1 provides a broad range of information designed to familiarize the reader with computer-generated ground motion maps, sources of strong-motion data and computer-generated ground-motion maps (including current web site addresses of principal providers). Chapter 1 also introduces current strategic planning for seismic monitoring statewide, including the goals for the next five years of the California Integrated Seismic Network[1] (CISN). The discussion notes that as efforts are undertaken over the next five years to meet these goals, as well as the

[1] The California Integrated Seismic Network is being proposed to provide the organizational framework to integrate the existing, separate monitoring networks in California into a single seismic monitoring system. The CISN Draft Strategic Plan for 2002-2006 includes the following goals: (1) operate a reliable and robust statewide system to record earthquake ground motions over the relevant range of frequencies and shaking levels; (2) distribute information about earthquakes rapidly after their occurrence for emergency response and public information; (3) create an easily accessible archive of California earthquake data for engineering and seismological research, including waveform data and derived products; (4) maintain CISN infrastructure as a reliable state-of-the-art data collection, processing, and information distribution system; (5) apply the latest research and technology to develop new algorithms for analyzing earthquake data and extracting more detailed information for new user products; and (6) maximize the use and benefit of real-time seismic information and other rapidly evolving tools and technologies through technology transfer to the user community.

goals of the proposed Advanced National Seismic System[2] (ANSS), the overview of strong-motion data resources in California provided in Chapter 1 is certain to be superceded by more current information. In general, it is noted that the efforts under the CISN and ANSS will provide additional resources and programs that will undoubtedly result in the more effective implementation of the *Guidelines*.

Chapter 2 covers procedures for using computer-generated maps for postearthquake response. The chapter begins with a section on the general framework for the use of real-time data for emergency response, including the data resources and procedures that are commonly related to the utilization of strong ground motion data for the various areas of emergency response. The subsequent sections review the particular interests and needs of the ten emergency response areas listed above. Real and hypothetical examples are included to illustrate the use of ShakeMap products in emergency response.

Procedures for using and interpreting strong-motion records to evaluate the postearthquake response of structures, including structures in or on which strong-motion instruments have been installed as well as non-instrumented structures, are described in Chapter 3. The chapter covers buildings, bridges, and dams. For each of these three structure type, the most commonly used procedures are described, including assumptions about structural properties, applicable structure types, minimum instrumentation and data required, steps to be taken, outputs, and example applications. For buildings, the *Guidelines* address:

- damage indicators that are sometimes evident in strong-motion data from instrumented buildings;
- procedures for rapid visual and hand-calculator analysis of strong-motion data from instrumented buildings (using data collected both at the ground level and in the upper stories, including perhaps film records);
- rapid estimation of changes in building period during strong ground shaking, using visual inspection and Fourier analysis techniques;
- rapid estimation of inter-story drifts, including estimates based on response spectra calculated for ground motion records and estimates based on displacement time-history analysis involving differencing of displacement time histories calculated from acceleration time-histories recorded at different story levels; and
- procedures to verify and define mathematical models of building behavior.

Similar procedures are provided for bridges and dams, but not in such detail.

Chapter 4 focuses on procedures for documenting structural attributes and performance in the vicinity of ground motion recordings. Similar to Chapter 3, this chapter covers procedures for buildings, bridges, and dams and provides guidance for both instrumented and non-instrumented structures. For non-instrumented buildings, the procedures draw heavily on the approach used after the 1994 Northridge earthquake to collect data on the characteristics and performance of

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[2] The Advanced National Seismic System Network, as currently planned, will be a nationwide network of at least 7,000 shaking measurement systems, both on the ground and in buildings (USGS, 2000).

more than 500 buildings within 1000 feet of strong-motion recording sites (see Figure 3). For each structure type, the steps for data collection, data formatting and archiving, and data analysis and dissemination are included.

A summary of the *Guidelines* is given in Chapter 5, along with an emphasis on the key recommendations for how the strong-motion maps and data can be effectively utilized for postearthquake response and evaluation of structures. The recommendations highlighted in this chapter, as well as in other sections of the *Guidelines*, are based on input from current and potential users obtained from interviews and the Users' Workshop. Additional input was provided by the project Resource and Advisory Panel.

Three appendices are included that contain supplemental information. Appendix A describes the process that was used to develop this document, Appendix B includes a summary of the most commonly used regional earthquake loss-estimation methods, and Appendix C includes a summary of the most commonly used linear and nonlinear structural analysis software programs.

References and a Glossary listing the acronyms and notation used in the document follow the appendices.

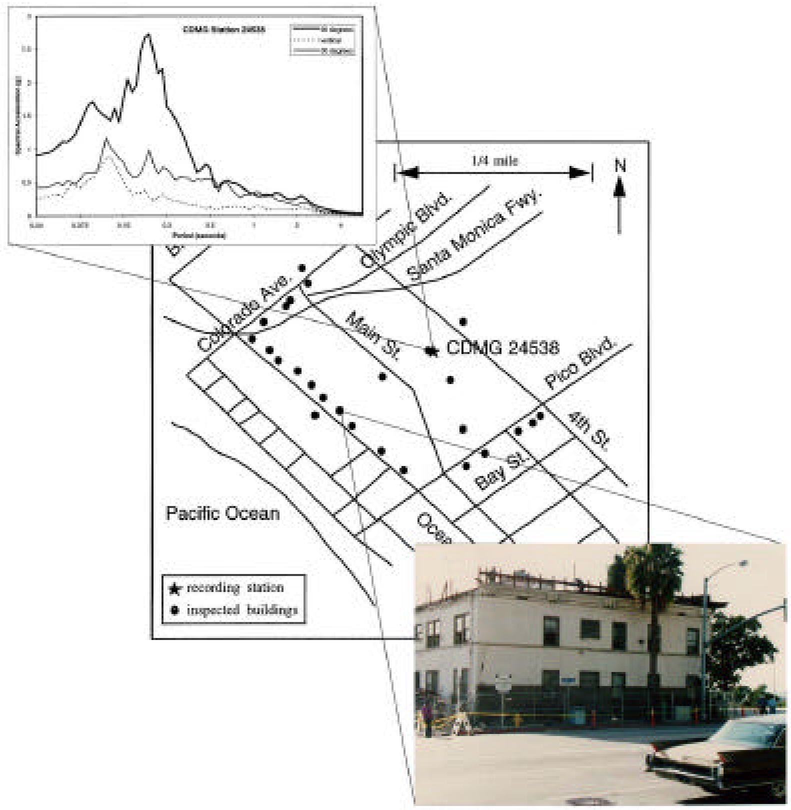
Preliminary Recommendations

Based on the efforts to date, the Project Team has developed the following preliminary recommendations for improving the use of strong-motion data and maps in postearthquake response planning and execution and postearthquake evaluation of structures:

1. For emergency response topics:
    - Develop or improve electronic databases containing facility information;
    - Convert information to GIS format and develop method for importing ShakeMap;
    - Automate simple damage and loss models based on specific post-event needs;
    - Consider use of maps that depict damage-potential ground-motion parameters (e.g., results from Bozorgnia's Year 2000 CSMIP Data Interpretation Project);
    - Automate the ranking of regions or facilities for response or inspection, respectively, if possible;
    - If already using loss-estimation software, improve databases for local regions;
    - Test system regularly;
    - Incorporate personal knowledge in all automated procedures to help convince personnel to start to trust computer output for first-order screening and ranking; and
    - Produce ShakeMaps at a larger scale, such as 1:15,000.
2. For evaluating data recorded in structures:
    - Develop pre-event computer models of the structure for the various types of analysis described;

ATC-38, Development of a Database on the Performance of Structures Near Strong-Motion Recordings

This project was formulated by ATC, the USGS, and several other northern California organizations immediately after the 1994 Northridge earthquake. The purpose was to document systematically the effects of the earthquake on structures adjacent to locations of strong ground motion recordings. Shortly after the earthquake, ATC dispatched teams of licensed civil and structural engineers to the areas of strong ground shaking to survey approximately 500 buildings in the vicinity of 30 strong-motion recording sites (within approximately 1000 feet) to document the characteristics and performance of buildings and other structures.



Prior to the site investigations, ATC provided training sessions to all inspectors instructing them on how to document their findings on the ATC-developed standardized survey forms, as well as in photographs. The data collected at each building site include information on the structure size, age, and location; the structural framing system and other important structural characteristics; nonstructural characteristics; geotechnical effects; performance characteristics; casualties; and estimated time to restore the facility to pre-earthquake usability. Damage is defined in both qualitative terms relating to repairability and in quantitative terms (estimated damage repair costs as a percentage of building replacement value). The survey data were archived in a relational database management system and mapped in a geographic information system. Digitized versions of the strong ground-motion recordings, as well as response spectra, for each site in the vicinity of which buildings were inspected, were also collected and archived with the survey data. The survey data and strong-motion information are documented in the ATC-38 Report and CD-ROM, *Database on the Performance of Structures Near Strong-Motion Recordings: 1994 Northridge, California, Earthquake*, which is available from the Applied Technology Council.

Figure 3. An Overview of the ATC-38 Database on the Performance of Structures Near Strong-Motion Recordings: 1994 Northridge, California, Earthquake (ATC, 2000).

- Develop means for quickly gathering and processing data; and
- Test system regularly.

3. For post-earthquake data collection:
   - Have procedures, personnel and funding ready before the event happens, including criteria for selecting facilities to inspect, and standardized data-collection forms;
   - Have computer database tables set up with trained personnel for data entry; and
   - Train inspectors in advance to collect the needed information using the standardized forms.
4. For strong-motion data and maps:
   - Increase density of instrumentation by:
     - Instrumenting additional buildings and other structures; and
     - Installing more free-field stations for improving ShakeMap interpolation between stations.

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GUIDELINES FOR UTILIZING SHAKEMAPS FOR POST-EARTHQUAKE RESPONSE

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Abstract

This paper describes portions of the document, *Guidelines for Using Strong-Motion Data for Postearthquake Response and Postearthquake Structural Evaluation*, currently being developed by the Applied Technology Council (ATC) for the California Division of Mines and Geology's (CDMG) Strong Motion Instrumentation Program (SMIP) 2000 Data Interpretation Project. The focus of this paper is on the use of computer-generated ground-motion maps, i.e., TriNet ShakeMaps, for post-earthquake response applications. Two companion papers presented at the SMIP02 Seminar, both by C. Rojahn et al., focus, respectively, on the overall description of the *Guidelines* and on the use of strong-motion data for structural evaluation. The procedures outlined in this paper are a summary of the information contained in the current draft of the document, which addresses ShakeMap applications for ten areas of post-earthquake response. The general framework is given here, with illustration for one application – damaged buildings and safety inspections.

Introduction

The development of the document, *Guidelines for Using Strong-Motion Data for Postearthquake Response and Postearthquake Structural Evaluation,* is a 2000 California Strong-Motion Instrumentation Program (CSMIP) Data Interpretation Project in progress by the Applied Technology Council (ATC) under contract to the California Division of Mines and Geology (CDMG). The primary objective of the *Guidelines* is to improve the state of the practice and facilitate improved emergency response and structural evaluation with the utilization of near real-time computer-generated ground-motion maps and strong-motion instrument recordings. The intended audience of the document includes emergency managers, contingency planners, government officials, risk managers, and practicing engineers.

The *Guidelines* focus on two primary topics. The first concerns the use of computer-generated ground-motion maps, such as TriNet ShakeMaps, in post-earthquake response. The intended use of this part of the document is to provide guidance on the development and implementation of applications using ShakeMap for emergency response. The second topic concerns the rapid utilization of near real-time instrumental recordings from ground and structure stations, so that the data will be particularly useful for post-earthquake response and evaluation of structures.

This paper is one of three papers presented at the SMIP02 Seminar that describe the in-progress development and anticipated contents of the document, *Guidelines for Using Strong-Motion Data for Postearthquake Response and Postearthquake Structural Evaluation*, to be published as the ATC-54 Report (ATC, in preparation). The focus of this paper is on the first of the two primary topics of the *Guidelines* discussed above – the use of computer-generated ground-motion maps, such as TriNet ShakeMaps, in post-earthquake response. The companion paper, "Guidelines for Utilizing Strong-Motion Data and ShakeMap Data in Post-Earthquake Response", by C. Rojahn, C.D. Comartin, and S.A. King provides an overview of the *Guidelines*, including the purpose, scope, development process, and contents. The second primary topic of the *Guidelines* – the use of strong-motion data for structural evaluation – is covered in the companion paper, "Guidelines for Utilization of Strong-Motion Data for Evaluation of Structures", by C. Rojahn, A.G. Brady, and C.D Comartin.

This paper begins with a description of computer-generated ground-motion maps, in particular the TriNet ShakeMaps. The procedures for using ShakeMaps in post-earthquake response are discussed next, including how the procedures were developed, general principles and guidelines, essential information and basic steps, and limitation on the use of ShakeMaps. The *Guidelines* address ShakeMap applications for approximately ten areas of post-earthquake response. Due to space limitations in this paper, the application description is limited to only one area – damaged buildings and safety inspections.

The material contained in this paper forms a portion of the *Guidelines* document, which is currently under development. The information has not yet been reviewed by intended users of the *Guidelines*, the CSMIP staff, the California Seismic Safety Commission's Strong Motion Instrumentation Advisory Committee (SMIAC), the ATC-54 Project Resource and Advisory Panel, or others, and as such should be considered preliminary and subject to revision.

Computer-Generated Ground-Motion Maps

In the portion of the *Guidelines* pertaining to computer-generated ground-motion maps, the focus is on ShakeMaps, produced by the TriNet program. TriNet is a five-year collaborative effort among the California Institute of Technology (Caltech), the United States Geological Survey (USGS), and the California Division of Mines and Geology (CDMG) to create an effective real-time earthquake information system for southern California and eventually northern California. A complete description of the history, background, and products of TriNet is available on the web site **www.trinet.org**. Most of the information described in this section is based on material contained in the ShakeMap section of the TriNet web site.

TriNet ShakeMaps are a representation of the ground shaking produced by an earthquake, an example of which is shown in Figure 1. They are generated automatically following moderate and large earthquakes. These are preliminary ground shaking maps, normally posted within several minutes of the earthquake origin time. They show the distribution of peak ground acceleration and velocity, spectral acceleration at three periods, and an instrumentally-derived, estimated Modified Mercalli Intensity. The Instrumental Intensity map is based on a combined regression of recorded peak acceleration and velocity amplitudes. In order to stabilize contouring and minimize the misrepresentation of the ground motion pattern due to data gaps, the data are augmented with predicted values in areas without recorded data. Given the epicenter and magnitude, peak motion amplitudes in sparse regions are estimated from attenuation curves. As the real-time TriNet station density increases, the reliance on predicted values will decrease.

In addition to producing near real-time ground shaking maps, the TriNet ShakeMap program also produces earthquake scenario ground shaking maps. The earthquake scenarios describe the expected ground motions and effects of specific hypothetical large earthquakes. The maps are used in planning and coordinating emergency response by utilities, emergency responders, and other agencies. The scenario earthquakes provide a more realistic example for training exercises and loss estimation studies, and can be generated for any hypothetical or historic earthquake. The steps involve assuming a particular fault or fault segment will (or did) rupture over a certain length, estimating the likely magnitude of the earthquake, and estimating the ground shaking at all locations in the chosen area around the fault. The ground motions are estimated using an empirical attenuation relationship, which is a predictive relationship that allows the estimation of the peak ground motions at a given distance and for an assumed magnitude.

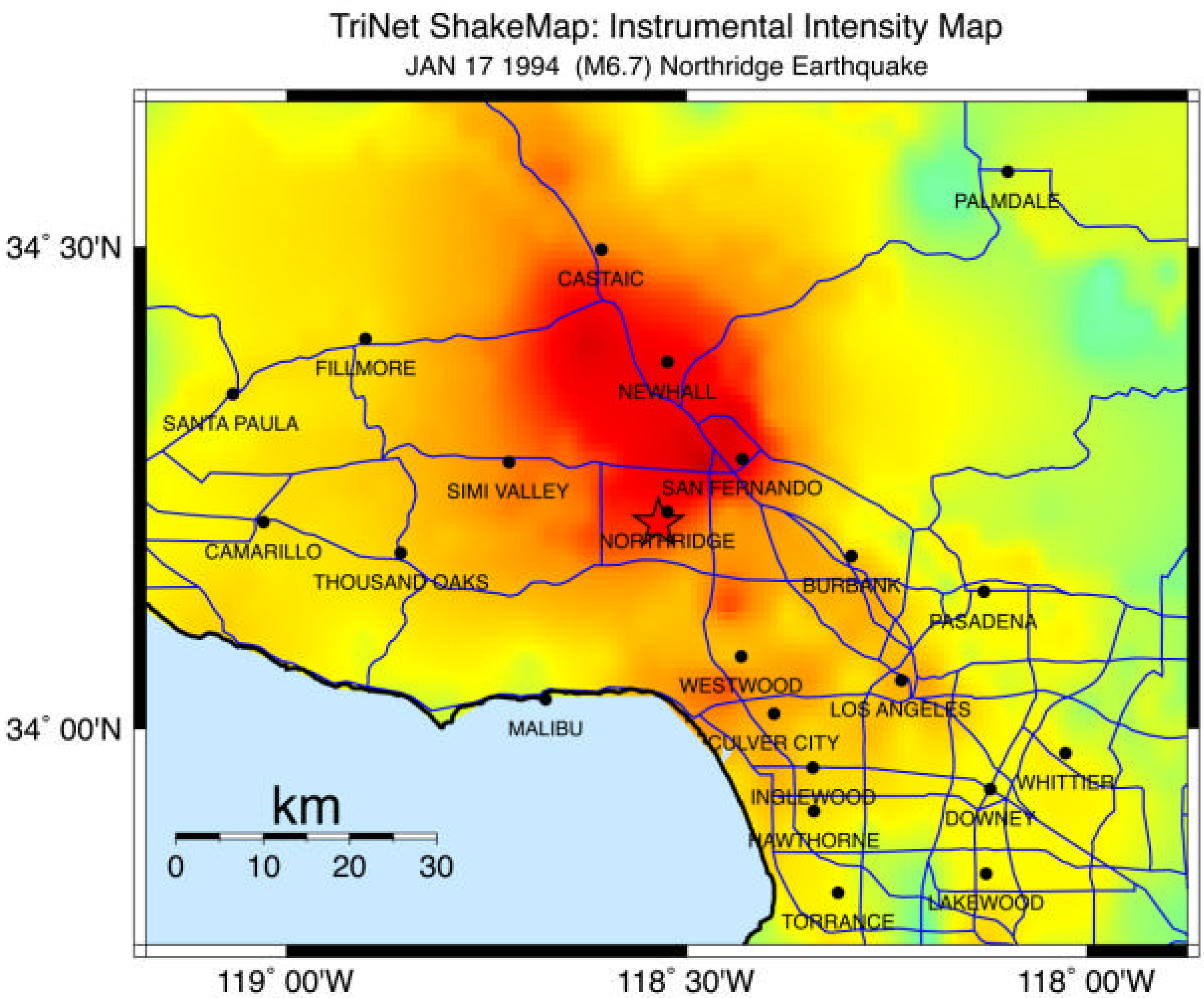
The web address for the TriNet ShakeMaps is **www.trinet.org/shake/**. Users of the *Guidelines* are encouraged to visit this site often, not only for the near real time ground shaking maps, but also for the new or improved products that are periodically added to the web site.

Procedure for Using Computer-Generated Ground-Motion Maps in Post-Earthquake Response

As discussed above, the *Guidelines* address the development and implementation of applications using ShakeMap for post-earthquake response, Specifically, the applications focus on the following post-earthquake response topics:

- extent of damaged buildings and planning related safety evaluation inspections
- condition of hospitals and other emergency response structures
- impact on utility systems and transportation networks
- extent of liquefaction, landslide, and inundation
- casualties and associated need for victim extraction from damaged structures
- extent of debris from collapsed structures
- sheltering needs
- extent of possible hazardous materials release
- preliminary economic loss estimates
- management of insurance claims

With respect to these applications, the *Guidelines* are intended to help users evaluate existing practices and policies, plan for future improvements, coordinate mutual aid, allocate resources, and design and budget for mitigation and planning exercises and programs.



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
PEAK VEL. (cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	8.1-16	16-31	31-60	60-116	>116
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Figure 1 *TriNet ShakeMap for the 1994 Northridge, California earthquake (image provided by David Wald, U.S. Geological Survey).*

Background

The *Guidelines* were developed through a multi-step approach, which is described in more detail in Appendix A of the document and in the companion paper by C. Rojahn, C.D. Comartin, and S.A. King presented at this seminar. The guidance provided on how to develop capabilities for using computer-generated ground-motion maps in post-earthquake response is the result of many months of effort by the project team members. They first identified and described the state-of-the-art in available data resources, building and lifeline inventory data, geographic information system (GIS) hazard maps, and loss estimation tools. Next, for each of the ten emergency response topics listed above, they defined the state-of-the practice at the state, regional, and local levels. Based on this information, primarily gathered through interviews with key individuals, an assessment was made of the existing capabilities in emergency response planning, i.e., how the identified available data resources are currently being used and how they might be utilized more effectively.

In the *Guidelines*, the procedures for using computer-generated ground-motion maps in post-earthquake response are described for each of the ten post-earthquake response topics. This information is prefaced by a section that outlines the general framework for use of near real-time data, covering material that is common to the ten areas of post-earthquake response. The general framework includes the essential information and basic steps, as well as the limitations to the ShakeMap applications, and is summarized below.

General Principles and Guidelines

There are several basic concepts related to the use of strong ground motion maps and data for post-earthquake response. The focus here is on emergency response – the decisions that are made immediately after an earthquake has occurred. Time and effective communication are critical, as the needs for quick and reliable decisions and information dissemination are typically the most important issues facing emergency managers. Given an earthquake occurrence, questions such as the following need to be immediately addressed:

- What has happened and where?
- How bad is it?
- How can I allocate my resources most effectively?

As discussed briefly in this paper and more thoroughly in the *Guidelines*, the use of near real-time ground-motion maps can provide information that helps answer these questions.

Essential Information

Near real-time ground-motion maps (i.e., TriNet ShakeMaps) provide excellent information on the distribution of shaking in the region affected by the earthquake. Post-earthquake response decisions can be made based only on the ground shaking information, however; these decisions require various levels of inference and are not making the most effective use of the ground shaking data. Combining the ground shaking information with other types of data for the region will allow for more reliable and meaningful emergency response decisions.

The basic information that is essential for making quick and reliable post-earthquake response decisions includes:

- Ground Shaking Data – information about the distribution of ground shaking in the region
- Facility Inventory Data – information about structures in the region
- Demographic Data – information about people who live or work in the region
- Vulnerability Data – information about how structures and people are typically affected by various levels of ground shaking

The most efficient procedure for storing, combining, and displaying these various types of data is through the use of a geographic information system (GIS). A GIS is similar to a regular database management system, except that in addition to dealing with tables of data, it has the added capability of storing and processing data on maps. Information on individual maps can be overlaid (or combined to form new maps) to show relationships and help with decision making, especially those that involve locations in a region.

A GIS with complete databases for a region is the ideal, but not often the reality, of those involved with post-earthquake response. The time and financial resources involved with setting up the system with required maps and data can be quite substantial, even for a small region. The procedures described in the *Guidelines* assume the most basic level of user in terms of experience and know-how, but not in terms of access to computer and data resources, as well as GIS or relational database management software. The purpose of the *Guidelines* is to outline the procedures for the most effective use of strong-motion data and maps, which in almost all cases involves combining the strong-motion maps and data with other types of data for the region.

Basic Steps

The basic steps for effectively using computer-generated ground-motion maps in post-earthquake response are outlined in this section. They are general, as the more specific information is described in the sections of the *Guidelines* that deal with the individual post-earthquake response topics. Ideally, some of these steps would be done before an earthquake occurs, or the entire process could be done as a training/planning exercise. The steps include:

1. Download the relevant ShakeMaps that illustrate the distribution of ground shaking parameters in the region.

2. Assemble the relevant inventory data, such as building portfolio information, Census data, street maps, and utility system maps, that can be overlaid or combined with the ShakeMaps to identify areas or facilities subjected to high levels of shaking.

3. Estimate damage or loss to regions or facilities based on the combination of ground shaking levels and inventory information. Some users will rely on a specific loss

estimation methodology or software for this step. The three most commonly used ones, HAZUS (NIBS, 1999), ATC-13 (ATC, 1985), and EPEDAT (Eguchi, et al, 1997), are described in Appendix B of the *Guidelines*.

4. Combine or overlay additional inventory data, such as emergency vehicle locations, shelters, and hospitals, as needed to provide information for decision making.

Limitations

There are several general limitations that should be kept in mind when using the computer-generated ground-motion maps for post-earthquake response. The most important issues include the following; more specific ones are discussed in the sections of the *Guidelines* that deal with the individual emergency response topics:

- ShakeMaps are generated automatically after moderate and large earthquakes and are not initially checked by humans. They are based on recorded data and augmented with predicted values in areas without a sufficient number of recording instruments. It is possible that the distribution of shaking will be biased towards a high anomalous recording, such as the Tarzana record in the 1994 Northridge earthquake.

- Following an earthquake, users need to be able to rapidly update data and mapped information based on reports from the field and revised ShakeMaps.

- Inventory data needs to be kept up to date in terms of accuracy and completeness, especially with respect to locations and facility information.

Application to Damaged Buildings and Safety Inspections

For each of the ten areas of post-earthquake response listed previously, the *Guidelines* describe the procedures for effectively utilizing ShakeMaps for post-earthquake response by discussing the typical users and needs, the potential data resources, and the potential models or data analysis procedures. Examples, real and hypothetical, are included to illustrate the concepts. The remainder of this paper summarizes the information contained in the *Guidelines* for one of the ten areas of post-earthquake response – damaged buildings and safety inspections.

Typical users and needs

Near real-time ground-motion data will be most useful in aiding engineers or officials in local jurisdictions with prioritizing building inspections within the first day or two following an event. In this application, the focus is on the use of ShakeMaps for help with making quick and reliable decisions, typically for a large group of buildings or for all buildings within a specific region. More advanced structural modeling for individual buildings using recorded instrumental data is covered in other sections of the *Guidelines*.

Following a moderate to large earthquake, a building owner or manager is under pressure from the occupants to have a trained professional inspect the building and determine whether or not it is safe to occupy. Owners and managers of multiple buildings, as well as the consulting engineers they hire for building investigation services, typically need some sort of priority ranking to effectively deal with occupancy decisions within a reasonable amount of time. Computer-generated ground-motion maps, such as ShakeMap, can be used to quickly determine the level of ground shaking experienced at each building and, when combined with structural and occupancy information, help illustrate which buildings should be inspected first.

An owner or manger of a single building is not likely to be interested in the ground shaking at the site, as this person will probably either call an engineer immediately after the event based on its magnitude and location or later after receiving reports of damage from the building occupants. Similarly, an owner or manager of several buildings clustered in a small region would assume that the ground motion is constant throughout the region, and would likely rely on an inspection priority scheme that relates only to building type and/or occupancy.

Local emergency response managers and building officials would use near real-time ground-motion maps to help prioritize the inspection of public and essential services buildings, as well as allocate staff or consultants for responding to citizen requests for assistance with building safety issues. In addition, this information could be used to notify residents or businesses about the potential loss of city services in specific areas, assign police and fire response to neighborhoods most likely to be damaged, establish the most critical locations to set up emergency shelters, and several other uses as described in the sections of the *Guidelines* focusing on these other applications.

Potential data resources

In order to effectively use computer-generated ground-shaking maps for prioritizing building inspections and determining regions of most severe damage, building information needs to be stored electronically and geographically referenced. Most building owners or managers have electronic databases of their facilities; however, few have this information in a geographic information system (GIS). As described previously, one of the basic analysis steps involves being able to overlay a map of facilities on the map of ground shaking distribution in the region. Converting existing electronic or paper building inventory databases to GIS format is not as difficult or time consuming as it would seem, given the user-friendly and reasonably-priced GIS software that is now available. In addition, the ability to store and manipulate building inventory data in a GIS has many benefits beyond responding to an earthquake.

Overlaying a map of buildings on a map of ground shaking distribution in the region will identify which buildings were subjected to the various levels of ground shaking. To make the most effective use of the GIS data and capabilities, the building data should include structural information, attributes that are often not part of typical building inventories. The exact structural information to be collected and stored depends on the resources available for database development (some information may require a structural engineer), as well as how the data are going to be used in the future, for post-earthquake response and other building management decisions. A relatively complete record in a building inventory database would include the following information:

- Location: address, ZIP code, Census tract, longitude and latitude
- Size: square footage, height, number of stories
- Construction data: year built, lateral load system, gravity load system,
- Occupancy data: use type, daytime occupancy, nighttime occupancy
- Other: existing condition, retrofits, irregularities, importance factor

The information listed above is sufficient in most cases to make first order estimates of earthquake damage and loss to buildings when combined with a map of ground shaking distribution. More detailed information on building attributes, such as that collected during rapid visual screening using ATC-21 procedures (ATC, 1988), results from detailed building evaluations using FEMA 310 (ASCE, 1998) or push-over analysis investigations to develop capacity curves, would provide an improved capability for estimating building vulnerability. Most building owners and managers are not likely to make the investment required to hire structural engineers to develop these data, as the cost versus the perceived benefit in automatically generating more detailed damage estimates for post-earthquake inspection is not readily apparent. They do, however, see the benefit in having engineers write reports on the structural quality of select critical buildings, and for the engineers to be available after an earthquake to use these reports in their damage assessment.

For regional use of computer-generated ground-shaking maps, building information is typically stored by summary statistics for the area. For example, Census tract or ZIP code maps can have the number or square footage of each building type as an attribute in the GIS database. The information is typically not very detailed because it is aggregated by geographic region and any building-specific information will be lost in the aggregation. Additionally, the use of the data for first-order prioritization of damaged areas, does not warrant more detailed building-specific information. Regional databases of building inventory can be found in existing loss estimation software or can be developed using techniques described in the loss estimation methodology reports. Information on loss estimation methods and software is described in Appendix B of the *Guidelines*.

Potential models or data analysis procedures

Building owners and managers typically rank life safety as the top priority and business operation as the next most important for prioritizing post-earthquake building inspections. In order to use near real-time ground motion information they must develop at least four important pieces of information before the earthquake occurs. These are similar to the four basic steps outlined previously, and include:

- A database of their facilities with information on occupancy and the importance to overall business operations.
- A list of engineers who are contracted to provide post-earthquake inspections. In lieu of this, companies will rely on building officials from the local jurisdiction to make inspections.
- A software program (typically a GIS) that can be used to access and store the near real-time ground motion maps and combine them with the facility database.

- Models that: (1) relate the level of ground shaking to damage and loss of function for each building (such as those found in the loss estimation methods described in Appendix B of the *Guidelines*), and (2) assign an inspection priority to each building (this is user-dependent). The level of sophistication of the models depends on the financial resources of the building owner or manager, the in-house technical capabilities, the level of detail in the facility databases, and the desired results. These models can include:
    1. Simple visual inspection of map overlays to make qualitative decisions
    2. Programs within the software that will do the analyses automatically
    3. Programs external to the software, run as a post-processor on the output of the map overlays

The information described above also applies to regional use by local emergency response managers and building officials. The main differences are in the facility databases as discussed above. In this case, the building information is stored in an aggregated format. Local officials are likely to be estimating building damage in conjunction with other effects of the earthquake, such as casualties, need for shelter, and preliminary economic loss – many of which are conditional on building damage. Although several of them still rely on manual methods as discussed in the *Guidelines*, the most efficient methods for making first-order estimates of emergency response needs in a region require the investment to develop accurate regional databases of facility information, and to acquire and learn an automated GIS-based loss estimation methodology.

Example

In the following example, a city and two building owners within the city cooperate to develop a post-earthquake response and recovery program. For this example, a city in southern California and two hypothetical companies (ABC, a high-technology company, and XYZ, a chain of grocery stores) are used. After an earthquake, the city's primary responsibility is to inspect its residential housing stock and the facilities it owns. A secondary but important goal is to make sure that businesses are adequately inspected. The purpose of both of these goals is two-fold: first, to insure that dangerous buildings are declared unsafe (red-tagged), and second, to allow safe buildings to be reoccupied.

ABC company has ten facilities in the area, located at four campuses. The campuses are primarily: manufacturing, research and development (R&D), office, and warehousing. A basic seismic study of the buildings has been done by the company's structural engineering consultant, and the estimated performance of each in a given Modified Mercalli Intensity (MMI) event is as shown in Table 1. The company has decided that a more detailed assessment of shaking intensity is not warranted for an initial response. They have prioritized the value of their buildings in the order shown in Table 1, and have decided that if the intensity at any facility exceeds the life safety threshold, that facility is inspected first. If more than one facility exceeds the life safety threshold, they are inspected in order of the number of occupants. Buildings in areas not exceeding the life safety threshold are inspected in the order shown in Table 1.

An earthquake strikes southern California with an intensity distribution as shown in Figure 2. (Note that this map is one of the ShakeMap Earthquake Planning Scenarios taken from the ShakeMap web site **www.trinet.org/shake/**.) Based on this intensity map and the location of the four campuses as shown, a simple GIS-based algorithm is developed to prioritize the inspections as: E, F, G, A, C, B, D, H, I, J. ABC uses this information to send its inspecting engineers to the building sites to make an ATC-20 (ATC, 1995) detailed evaluation, suitable for posting the buildings as red-tagged (unsafe), yellow-tagged (restricted use), or green-tagged (inspected).

Table 1 Estimated Seismic Performance of Buildings in Example

		Performance Threshold at MMI:	
Campus	**Building**	**Functionality**	**Life Safety**
Manufacturing	A	VI	VIII
	B	VII	IX
R&D	C	V	VII
	D	VI	VIII
Office	E	VII	IX
	F	VII	IX
	G	V	VII
Warehousing	H	VII	IX
	I	VII	IX
	J	VI	VIII

A week following the initial posting, ABC's engineers determine that buildings C, D and F are susceptible to structural damage that was not evident from an initial walkthrough of the building. ABC's engineers use pushover curves (estimates of the capacity of each building) developed prior to the earthquake with the response spectra obtained from ground motions recorded at nearby free field instruments. They are then able to determine if and where damage may be concentrated and respond accordingly.

XYZ company is unable to contract with structural engineers because of the lack of financial resources. The company has ten structures spread throughout the area and will rely on city inspectors to evaluate the facilities. The company develops a cooperative relationship with the city as follows: XYZ supplies the city with a list of its buildings, photographs, a brief description of the number of stories, year of construction, and material type, and any structural drawings it may have, reduced to 11x17 format.

The city creates a GIS map of the residential and public facilities with an intensity-based inspection prioritization similar to ABC company's. It then runs several scenario earthquakes through a model that creates estimated intensity contours. Based on these scenarios, the city determines its immediate inspection needs for the housing and public building stock. It estimates how long these inspections will take, and places XYZ company on a waiting list for inspection after the initial inspections are completed. The city then gives XYZ company an estimate of how long it will take to have its buildings inspected after each scenario. Because the city has basic information on the buildings, provided by XYZ, it is theoretically able to make inspections of XYZ's buildings more quickly and accurately. The city also determines for each scenario, which of XYZ's buildings are likely to fall outside the contours of shaking intensity that would cause moderate to severe damage.

An earthquake occurs and the near real-time map of ground shaking intensity generated from free field instrumentation is incorporated into the city's GIS software as shown in Figure 3. The city notifies XYZ company that it will take approximately 96 hours for inspectors to get to XYZ's buildings, but it also tells them that four of their ten buildings are not within the high intensity ground shaking zone, and unless hazardous damage is clearly evident the buildings can be occupied.

The use of recorded ground motion in the above example is threefold. First, on a near real time basis, the general distribution of intensity is used to make a rapid prioritization of inspection needs, both for the city and for ABC company. Near real time information would not typically be used in this case to make a determination or estimation of specific building damage. Second, more detailed ground motion information that would not have to be assembled in real time would be used in the days following the event, to help engineers analyze building damage. Third, companies are able to get estimates quickly after an event of when their buildings will be inspected, and whether or not certain buildings outside the zones of high shaking can be reoccupied immediately.

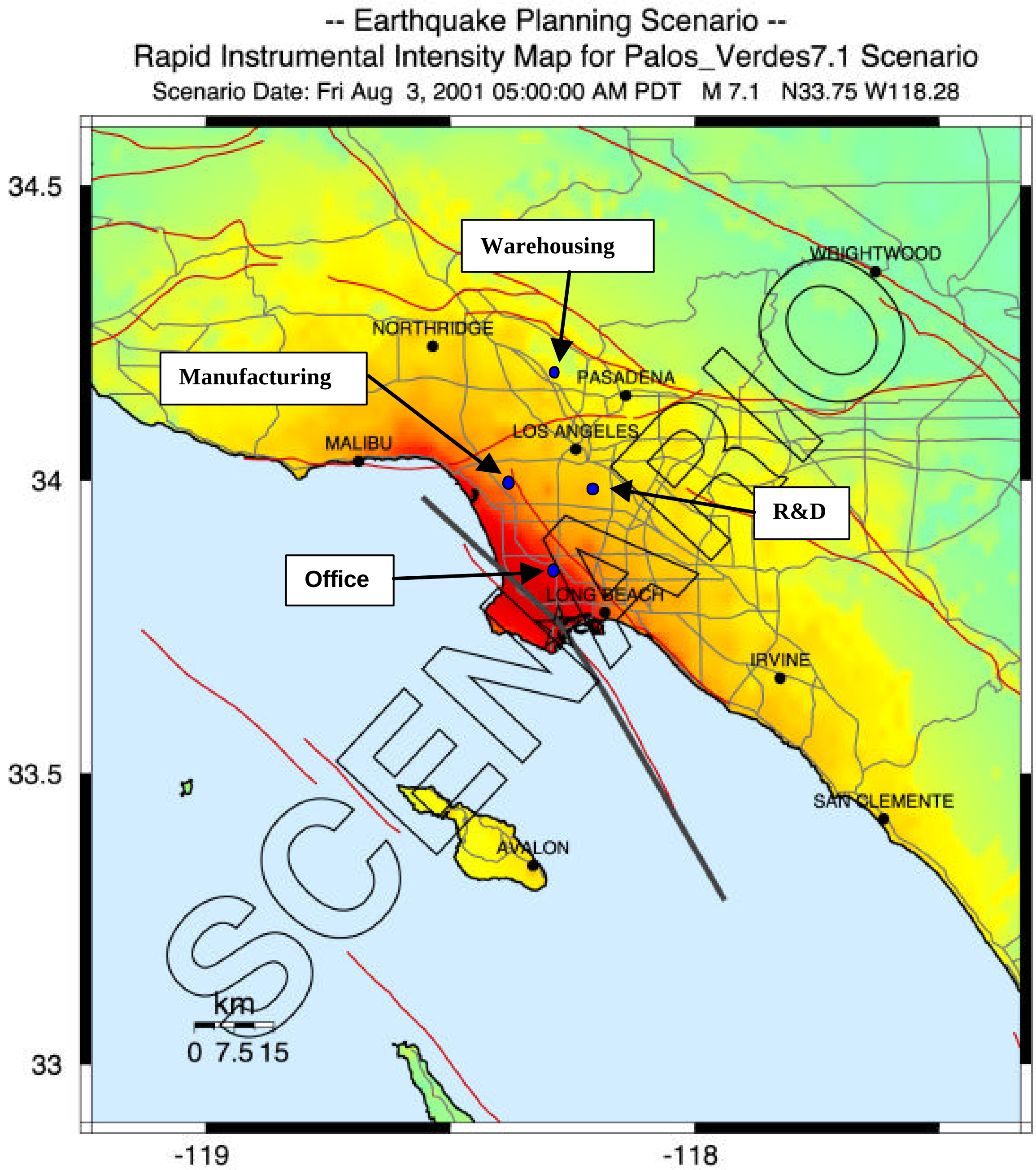
Summary

This paper describes one of the key topics of the document, *Guidelines for Using Strong-Motion Data for Postearthquake Response and Postearthquake Structural Evaluation*, currently being developed by ATC for CDMG as one of the 2000 CSMIP Data Interpretation Projects. It concerns the development and implementation of applications using computer-generated ground-motion maps, such as TriNet ShakeMaps, for post-earthquake response. The *Guidelines* address ShakeMap applications for approximately ten areas of post-earthquake response. In this paper, the focus is on the general framework, including essential information, basic steps, and limitations. Due to space limitations, the application description is limited to only one of the ten emergency response topics – damaged buildings and safety inspections.

It should be emphasized again that the material in this paper forms a portion of the draft *Guidelines* document, and should be considered preliminary until the final document is released.

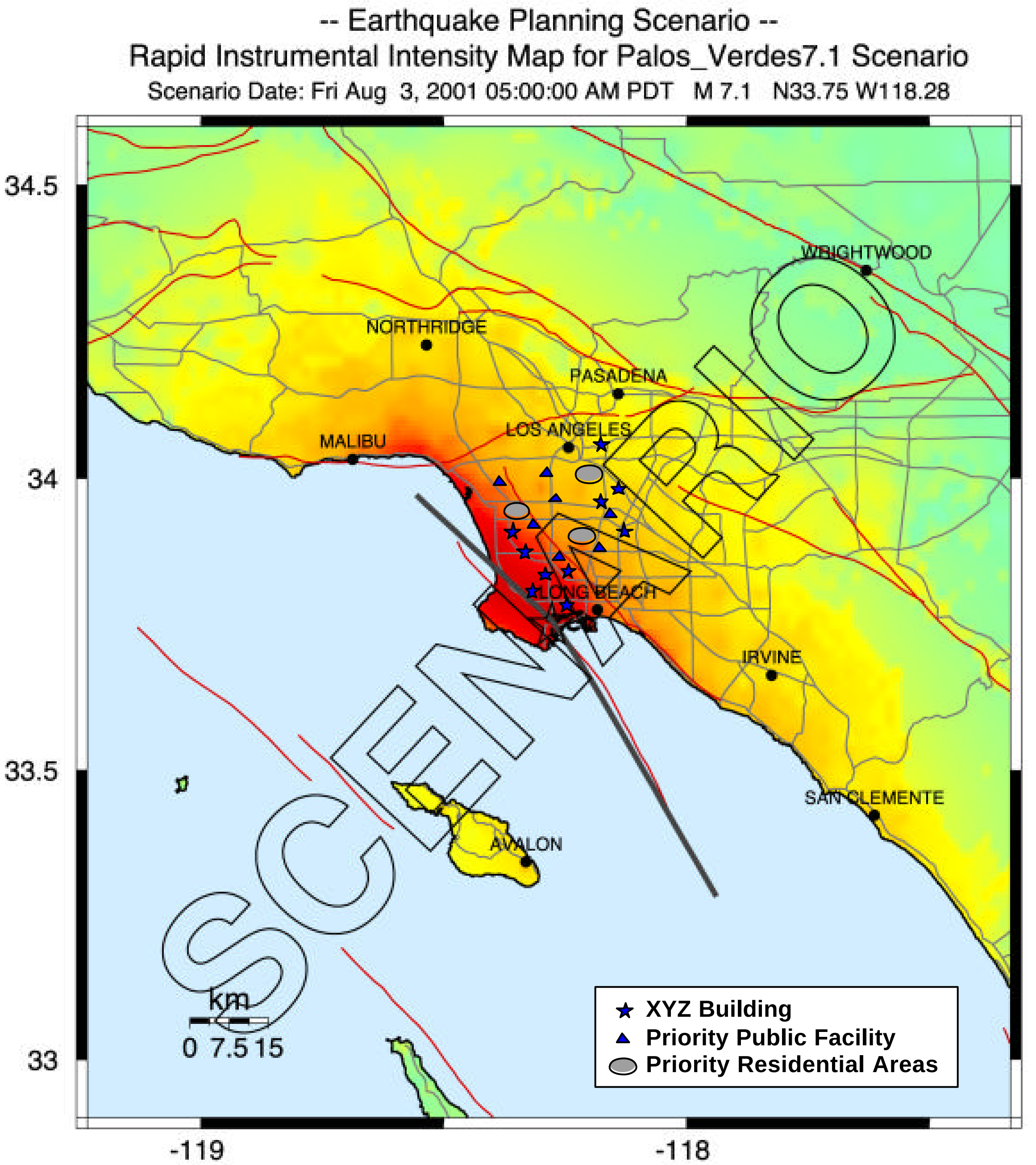
Acknowledgements

Several people have contributed to the development of the material contained in the *Guidelines* pertaining to the use of ShakeMap for emergency response, a subset of which is contained in this paper. In particular, we wish to acknowledge the input of David Wald of the U.S. Geological Survey, James Goltz of the California Institute of Technology, several members of the California Seismic Safety Commission's Strong Motion Instrumentation Advisory Committee, the ATC-54 Project Advisory Panel, and the numerous individuals in the user community who were interviewed by members of the project team.



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Moderate/Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<.17	.17-1.4	1.4-3.9	3.9-9.2	9.2-18	18-34	34-65	65-124	>124
PEAK VEL. (cm/s)	<0.1	0.1-1.1	1.1-3.4	3.4-8.1	8.1-16	16-31	31-60	60-116	>116
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Figure 2 *Distribution of ground shaking intensity with location of example ABC Company buildings.*



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
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INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

Figure 3 *Distribution of ground shaking intensity with location of example XYZ Company buildings.*

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GUIDELINES FOR UTILIZING STRONG-MOTION DATA FOR POST-EARTHQUAKE EVALUATION OF STRUCTURES

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Abstract

This paper describes portions of the document, *Guidelines for Using Strong-Motion Data for Postearthquake Response and Postearthquake Structural Evaluation*, currently being developed by the Applied Technology Council for the California Geological Survey (formerly, California Division of Mines and Geology) Strong Motion Instrumentation Program 2000 Data Interpretation Project. The focus of the paper is on guidance for using strong-motion data to evaluate the performance of structures in the immediate earthquake aftermath. Topics addressed include strong-motion data sources and processing, damage indicators that can often be observed in recorded data, and methods for rapid interpretation of strong-motion data to evaluate structural performance.

Introduction

Instrumentation in current strong-motion networks in California has reached different stages along the steady technological upgrade from photographic paper recording to digital recording, with the accompanying improvement in the availability of data from a delay of several months or more, to a delay of only seconds after a seismic event. Instruments now in place record either analog film records or digital data, with the trend toward exclusive use of instruments that record digital data. In California the number of free-field sites, buildings, and other structures containing strong-motion instruments has expanded to more than 1000 since the original installation of nine strong-motion instruments in California in 1932. Today, the large number of instrumented sites and the rapid availability of digital data, including analog data converted to digital format, provide the real potential for using strong-motion data to evaluate the performance of buildings and other structures in the immediate earthquake aftermath. The use of strong-motion data for this purpose is, in fact, the original justification for code-required instrumentation that began to be mandated after about 1965 for most buildings over six stories in height in certain California cities like Los Angeles, which adopted the appendix to Chapter 23 of the 1967 *Uniform Building Code*.

Methods and techniques for evaluating the performance of buildings and other structures using strong-motion data have been available for several decades, but widespread use has been hampered for several reasons: (1) until recently, the delayed availability of data in digital format, and (2) the lack of concise descriptions of such methods readily available to structures' owners and their engineers. To help remedy this situation, the California Division of Mines and Geology (CDMG) awarded a Year 2000 California Strong-Motion Instrumentation Program

(CSMIP) Data Interpretation Project to the Applied Technology Council (ATC) to prepare guidelines on this subject. The guidelines now under development by ATC address not only postearthquake structural evaluation, but also the use of computer-generated ground-motion maps in postearthquake response and the collection of data on the performance and characteristics of structures located in the vicinity of strong-motion recordings sites, or in which strong-motion instruments have been installed. The results of the ATC effort will be published in the ATC-54 report, *Guidelines for Using Strong-Motion Data for Postearthquake Response and Postearthquake Structural Evaluation* (ATC, *in preparation*).

In this paper we focus on those portions of the ATC-54 report pertaining to guidance for using strong-motion data for postearthquake evaluation of structures. We begin with background information that in part summarizes the results from our literature review to determine the extent to which strong-motion data have been used to date to evaluate structures. The background information also identifies and briefly discusses (1) the general limitations of strong-motion data for the purpose of evaluating structures; (2) the principal operators of existing strong-motion networks; (3) available resources for strong-motion data; and (4) data-processing issues. The intent is to provide users of those portions of the ATC-54 *Guidelines* pertaining to structural evaluation, namely, facility owners and their engineers, with information that will enable them to understand the limitations of strong-motion data, as well as the means by which such data can be acquired and processed. The principal focus of this paper is the description of methods for rapid interpretation of strong-motion data in order to evaluate structural performance. A discussion of these methods is preceded by a discussion on damage indicators that can often be detected by visual inspection of strong-motion analog data. Due to space limitations we focus on methods for buildings, the type of structure for which such methods are best developed. The companion paper by Rojahn et al. (2002), "Guidelines for Utilizing Strong-Motion Data and ShakeMaps in Postearthquake Response, An Overview", describes the contents of the ATC-54 *Guidelines*, and how the report was developed. The companion paper by King et al. (2002), "Guidelines for Utilizing ShakeMaps for Emergency Response", provides more in-depth information pertaining to the use of ShakeMaps. Similar papers by Rojahn et al. (2001) and King et al. (2001) were also developed for the SMIP01 Seminar scheduled for September 2001 in Los Angeles.

Background

Prior Efforts Using Strong-Motion Data for Structural Evaluation

The great majority of relevant papers in the literature describe research that used data that was available anywhere from several minutes after an earthquake event to several months afterwards, even though the research itself was completed, and reported on, up to several years later. In large part, the technical literature indicates that strong-motion data have been used in complex system identification studies, and in the identification of potential global structural damage under various levels of shaking intensity. There have also been many instances where strong-motion data have been used as input in computer analysis programs to calculate structural component forces, moments, displacements, and rotations.

Interestingly enough, we could find no examples in the technical published literature of the analysis of a recording that led to an assessment of damage, that, alone, resulted in a decision or action that saved lives or property threatened by such damage. We are aware, however, as a result of interviews with various researchers and program managers, that there have been instances where strong-motion recordings have alerted building or structural personnel that steps should be taken to improve structural performance, or that a safety check should be run because ground accelerations were higher than code values. These have not been reported in the technical literature due to the sensitive nature of the situation. They include: (1) the transfer of occupants from commercial buildings so that the buildings could be demolished and rebuilt, (2) the temporary shutting down of a power-generating plant for inspection due to ground accelerations higher than design code levels, and (3) retrofitting a government office building with damping devices after records confirmed that modal oscillations continued for longer durations than expected.

Limitations and Uses of Strong-Motion Data for Structural Evaluation

For complex structures, such as bridges, dams, lifeline structures, and buildings of complex shape, strong-motion recordings of seismic response have been used for research on system identification, in order to improve computer models of the structure. Such models include the mass and stiffness of structural components, if it is a linear model, and yield points and ultimate strengths, if it is a nonlinear model. Computer modeling of instrumented structures is a prerequisite if a detailed time-history analysis of linear or nonlinear structural response is contemplated, with a view to searching for the existence and extent of damage, directly from the records.

Most structures have separate structural components whose nonlinear behavior is difficult to model and is difficult to determine from a system identification analysis. Therefore, a rapid time-history analysis that uses all of the available strong-motion records of structural response, in conjunction with a computer model of the structure, is unlikely to reveal the precise location of local component degradation or serious damage.

Global structural degradation damage parameters, however, such as total horizontal drift at the roof level, or permanent lengthening of the fundamental period (first mode) of vibration, can be determined from a rapid time history analysis, with or without requiring a computer model of the structure. With a computer model, a best-fit matching of the structural strong-motion recordings with computed structural recordings, can be used to verify or calibrate an existing computer model, which can then be used to provide estimates of inter-story drift time-histories throughout the height of the structure, as well as information on period lengthening. Without a computer model, inter-story drift can be determined by differencing selected displacement time histories of structural response (computed by double integration of acceleration time histories), and estimates of the fundamental mode shape. As described later in this paper, the lengthening of the first natural period of vibration can also be determined visually from plots of acceleration time history response as well as Fourier analysis of strong-motion records using recognized moving-window procedures.

Assuming the mass of a building remains the same, the relationship between building stiffness and building period is defined by the following equation:

$$w = \sqrt{\left(\frac{k}{m}\right)}, \quad T = 2p\sqrt{\left(\frac{m}{k}\right)}, \qquad (1)$$

Based on this relationship, for example, a period lengthening of 25% in a frame building, indicates a drop in stiffness to 64% of its original linear value.

In the years following the damaging 1989 Loma Prieta, 1994 Northridge and 1995 Kobe earthquakes, structural instrumentation projects have concentrated more specifically on the search for damage assessment capability. A thoroughly instrumented building has more potential for the determination of the general location of damage, but not precise locations. The appearance of bursts of high-frequency acceleration in adjacent structural records is an indicator of possible local damage, but could also be the recording of elevated contents or nonstructural components falling to the floor.

Existing Strong-Motion Networks

The major strong-motion networks operating in California include structural instrumentation of various complexity. The three larger networks are: (1) the California Strong-Motion Instrumentation Program, California Geological Survey, Department of Conservation, operating throughout California and headquartered in Sacramento (Shakal et al., 1989b); (2) the National Cooperative Strong-Motion Instrumentation Network, U.S. Geological Survey, (USGS), headquartered in Menlo Park, California, and operating throughout the nation, and (3) the Southern California Instrumentation Network, headquartered at the University of Southern California (USC), and operating throughout the Los Angeles region. Smaller networks, which also include structural instrumentation, include the Army Corps of Engineers, the U.S. Bureau of Reclamation (Viksne et al., 1993), the Metropolitan Water District of Southern California, and the Los Angeles Department of Water and Power.

The instrumentation networks of CSMIP and the USGS contain ground-motion stations, instrumented buildings and other instrumented structures. The number of channels at a ground-motion station is generally three. The number of channels in a specific structure depends on the time it was first instrumented and the vibrational questions for which answers are sought, ranging from perhaps six to several dozen or more. The average is approximately sixteen. The instrument coverage in Los Angeles, in San Francisco, in California, on the Pacific coast, and across the nation ranges widely, but the percentage of the three different station types providing useful records from an urban earthquake can be judged from the numbers from the 1994 Northridge earthquake, namely, 250 ground-motion stations, 400 buildings, and 50 other structures.

Almost all instruments now record digitally at a sample rate between 50 and 200 samples per second on each channel. Often, the sensor has a wide frequency range, and the data are shared with researchers from the strong-motion seismology field. If data communication is possible between the recorder and the network central headquarters, and is not damaged nor interrupted by the earthquake, then data can be transmitted to the central headquarters for processing and dissemination to people needing to make rapid decisions. For those recorders without communication to central headquarters, a technician recovers the digital data during the days following the earthquake.

Data Sources and Processing

The technical literature and world wide web contain numerous resources describing existing strong-motion data and data processing techniques. In some instances the resources contain information on a wide variety of strong-motion recording sites and data, including both free-field sites and instrumented structures; others refer primarily to data sets from specific types of structures. Two of the most popular current resources are the COSMOS Virtual Data Center (db.cosmos-eq.org), which is operated by the Consortium of Organizations for Strong-Motion Observation Systems, and the Pacific Earthquake Engineering Research Center (PEER) Strong-Motion database (peer.berkeley.edu/smcat). The COSMOS site provides links to nearly all of the major strong-motion data providers and is a well-designed web site for quickly identifying available strong-motion data, as well as listings of current stations.

The literature also contains numerous examples of studies and descriptions of strong-motion data sets from instrumented buildings, bridges, and dams. Citations for data from buildings, for example, include Shakal et al. (1989a), Reichle et al. (1990), Huang et al. (1991), Graizer et al. (1998), Porcella and Switzer (1989), Archuleta et al. (1999), the proceedings of the annual CSMIP seminars: Huang et al. (1992, 1993, 1994, 1995), and descriptions of data from specific events, such as described by Darragh et al., (1995) and Shakal and Huang (1995) for the 1994 Northridge earthquake. The instrumentation and records from a 2540-foot-long interchange bridge excited by the 1992 Landers and 1992 Big Bear earthquakes have been described by Huang and Shakal (1995). Similarly, significant earthquake strong-motion accelerograms recorded on or near dams, and current developments in instrumentation, including near-real-time strong-motion recording, are discussed in Shakal and Huang (1996).

With improvement in the instrumentation, and the increase in magnitude of the earthquakes producing the records, there is less need for elegant processing to push the envelope of useful data beyond that envisioned by the instrument manufacturer. The recording instruments and processing techniques now used by the principal providers of data (CDMG, USC, and USGS) provide strong-motion data that exclude stray digital points and incorporate (1) a high-frequency limit defined by both the sampling rate and the instrument response characteristics at high frequencies (for which corrections are made), and (2) a long-period limit defined by the signal-to-noise ratio at long periods and the duration of both the strong shaking and the total record (the larger the amplitudes over a long duration, the longer will be the selected cut-off period). Digital instruments also provide a pre-event memory capability to ensure that earthquake acceleration, calculated velocity, and calculated displacement all commence with zero amplitude.

Damage Indicators Evident in Strong-Motion Structural Records

As described below, certain characteristics evident in strong-motion records of ground motion or structural response may be indicative of structural damage. The appearance of these characteristics may warrant further inspection and evaluation of the structure for structural damage by a structural engineer experienced in seismic design.

1. Acceleration Response Exceeding Code Design Values

Design code values of effective peak ground acceleration, or acceleration response, are used in the design process. The distribution of forces along the vertical axis of the building is normally defined by an equation provided in the code, and the building is designed as if these distributed forces (a constant acceleration times mass) were acting at the various floor and roof levels. Maximum acceleration spectra computed using basement, ground level, or nearby free-field can be compared to design values to determine if the ground motions experienced by the structure exceeded those used in design. Similarly, design acceleration time histories recorded in the upper stories can be used to estimate base shear values, which can then be compared to those used in design. This process requires careful consideration of design assumptions, including assumed material strengths (e.g., ultimate strength versus working stress design) and parameters, such as R values and k value, used in the design process. The data interpretation process also requires careful consideration of the site soil conditions and the frequency characteristics of the input ground motion. For example, at high frequencies, the velocities and displacements have low amplitudes, and the structure may resist with ease apparent high amplitude motions (e.g., a sinusoidal acceleration amplitude of 0.5 *g* at 10 Hz produces a displacement amplitude of 0.05 inches). Guidance on these issues is provided in the ATC-54 *Guidelines*.

2. Lengthening of the Modal Periods

The lengthening of modal periods has been described by several researchers, including a summary by Naeim (1996) of work investigating the response of 20 CDMG-instrumented buildings during the 1994 Northridge earthquake. This summary includes buildings whose periods lengthened during the response. In a later work on the same 20 buildings, Naeim (1998a) observes that the second and third modes of vibration contribute significantly to the overall response in the cases of two buildings of 13 and 20 stories.

It has been known since the 1971 San Fernando earthquake that a lengthening period can be associated with damage to some extent. The increase in period can be visually read in a record of sufficient clarity containing a single mode response, or can be studied analytically with Fourier moving-window analyses (see Figure 1). For a structure shown later to have no structural damage, the lengthening of the period is small, and the period returns to its original value before the end of the record. Structural damage, on the other hand, lengthens the period to greater levels, and the period does not return to its original value. Rezai et al. (1998) analyzed the important San Fernando and Northridge records from a 16-channel recording system installed in a 7-story building. During the San Fernando earthquake, structural damage was minor. During Northridge, the period lengthened by 66% in one direction, and by 100% in the other. These percentages correspond to drops in stiffness to 0.36 and 0.25, respectively, of original values. Major structural damage occurred, and it is clear, from the lengthened period on the record, when this damage commenced.

Based on the above it is apparent that period lengthening indicates a decrease in stiffness. Separation of partitions from structural framing and structural walls, separation of structural

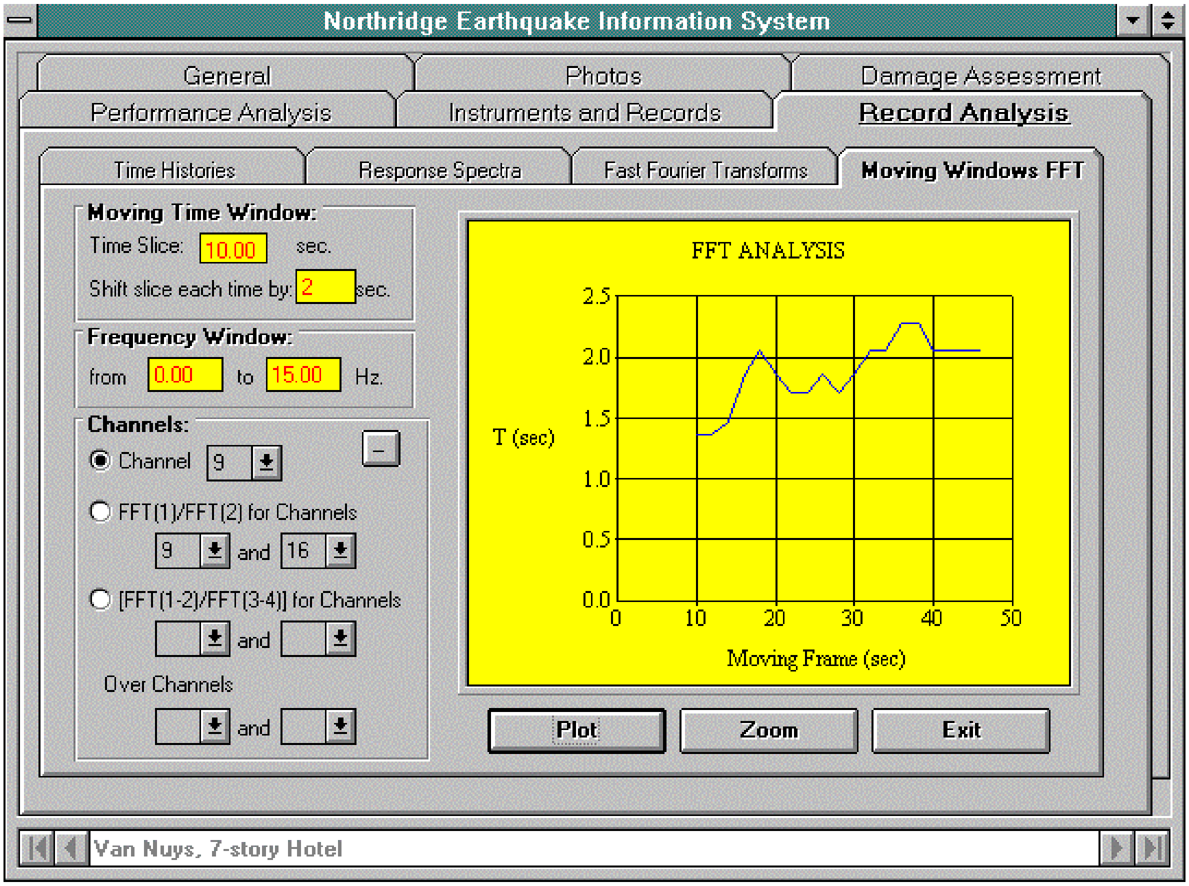

Figure 1. Moving window Fast Fourier Transform analysis showing significant softening of a 7-story hotel in Van Nuys, California, during strong ground shaking (from Naeim, 1998b).

infill from structural framing, and structural damage to seismic-resisting elements may contribute to this decrease in stiffness. Guidance on these issues is provided in the ATC-54 *Guidelines*.

3. High-Frequency Bursts of Acceleration

Damage occurring in a structure introduces impulsive forces created by the initial damage to steel and its connections, concrete and its reinforcing, masonry and its reinforcing and mortar, or wood and its connectors, followed by more impulsive forces created as damaged areas continue to impact on themselves as the response to the earthquake proceeds. The resulting compression and shear waves travel throughout the structure almost instantly. Those with audible frequencies are not recorded on the typical strong-motion recorder (these frequencies are too high) but waves with frequencies within the range of instrumental response, that is, up to 50 Hz or 100 Hz, are sensed by such recorders. The presence of a specific frequency in this range can be identified on an analog film record, but identification from a digital record depends on its sampling rate.

Damage may be indicated if a high-frequency burst of acceleration is identified on the record from several closely-spaced transducers, the high-frequency bursts continue to occur as time progresses, and it is evident that at the same time, modal periods lengthen and stay that way until the end of the record. An example of such a condition is shown in the records from the Imperial County Services Building (see Figures 2 and 3), which was extensively damaged by the 1979 Imperial Valley earthquake. It is necessary to distinguish this damage indicator from other possible causes, such as the falling to the floor of heavy elevated contents in the vicinity of a transducer. High-frequency bursts from such a cause will not be seen on distant recordings elsewhere in the structure, and will not be accompanied by lengthening periods. Guidance on this issue is provided in the ATC-54 *Guidelines.*

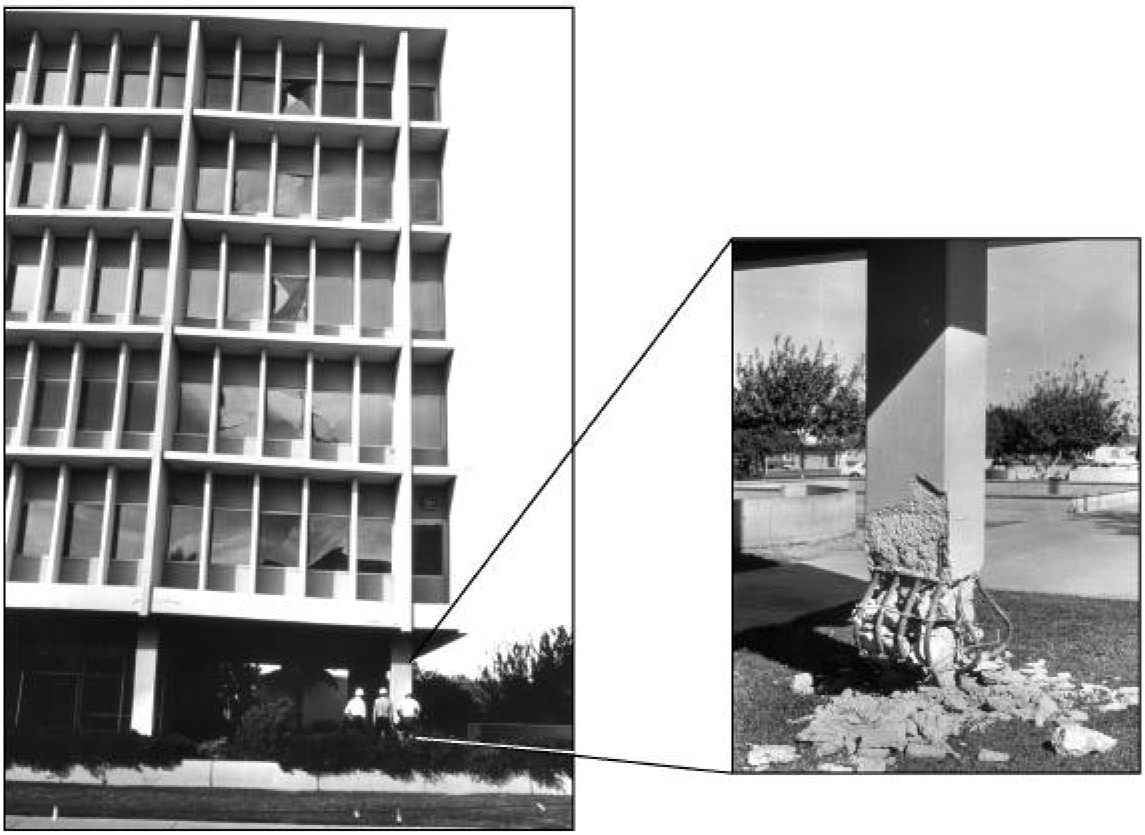

Figure 2. Photos of south face (left) and failed reinforced concrete column (right) at base of Imperial County Services Building, which was damaged by the 1979 Imperial Valley Earthquake. The burst of high frequency motion occurring simultaneously with column failure is shown in Figure 3 (from Rojahn and Mork, 1982).

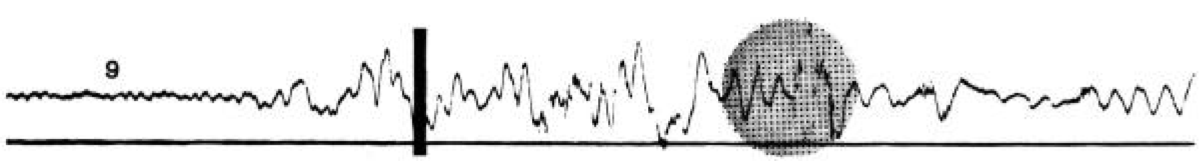

Figure 3. Copy of north-south analog acceleration time history recorded in North-South direction on second floor level (Trace 9), directly above the failed reinforced concrete column shown in Figure 2. Burst of high-frequency motion is shown in shaded circle. Vertical bar represents the time when the onset of damage is inferred to have occurred (from Rojahn and Mork, 1982).

Procedures for Using Strong-Motion Data to Evaluate Structural Response

Chapter 3 of the ATC-54 *Guidelines* provide step-by-step procedures to evaluate the response of structures (buildings, bridges, and dams) using recorded strong-motion data. For each type of structure, a variety of approaches are described, including assumptions about structural properties, applicable structure sub-types, minimum instrumentation and data required, steps to be taken, outputs, and example applications. It is assumed that the user is unfamiliar with strong ground-motion recordings. Also, a summary of available structural analysis programs and their capabilities for assisting in the interpretation of strong motion data is provided in an appendix of the *Guidelines*.

Procedures described in the ATC-54 *Guidelines* include those for:

- rapid estimation of changes in building period during strong ground shaking, using visual inspection and Fourier analysis techniques;
- hand modal analysis of strong-motion data from instrumented buildings to estimate maximum shear forces at the base of the structure (briefly summarized in Rojahn et al., 2001);
- rapid estimation of inter-story drifts, including estimates based on displacement time-history analysis involving differencing of displacement time histories calculated from acceleration time-histories recorded at different story levels;
- rapid estimation of maximum roof displacement using available methods for the design of building structures using nonlinear static analysis procedures, such as the Coefficient Method described in the FEMA 273 *NEHRP Guidelines for the Seismic Rehabilitation of Buildings* (ATC, 1997); and
- procedures to define and verify mathematical computer models of building behavior.

For illustrative purposes we briefly describe results from an analysis of strong-motion data from an extensively instrumented, severely damaged building to obtain estimates of inter-story drift.

Estimate of Inter-Story Drifts in the Imperial County Services Building During the 1979 Imperial County Services Building

Within days after the occurrence of the 1979 Imperial Valley, California, earthquake of October 15, 1979, researchers at the U. S. Geological Survey and other institutions commenced the process of analyzing strong-motion records from the severely damaged Imperial County Services Buildings (Figure 2), which contained an array of 13 accelerometers (Figure 4). By double integration of the acceleration time histories, and careful filtering of the data, it was possible to prepare displacement time histories of the response of the structure. By differencing the calculated displacement time histories (subtracting one from another) from horizontal transducers located at the roof, second floor, and ground levels and plotting the results on a single plot, it was possible to estimate drifts between selected floor levels, which could then be used as a basis for estimating inter-story drift. The results from this effort are shown in Figure 5.

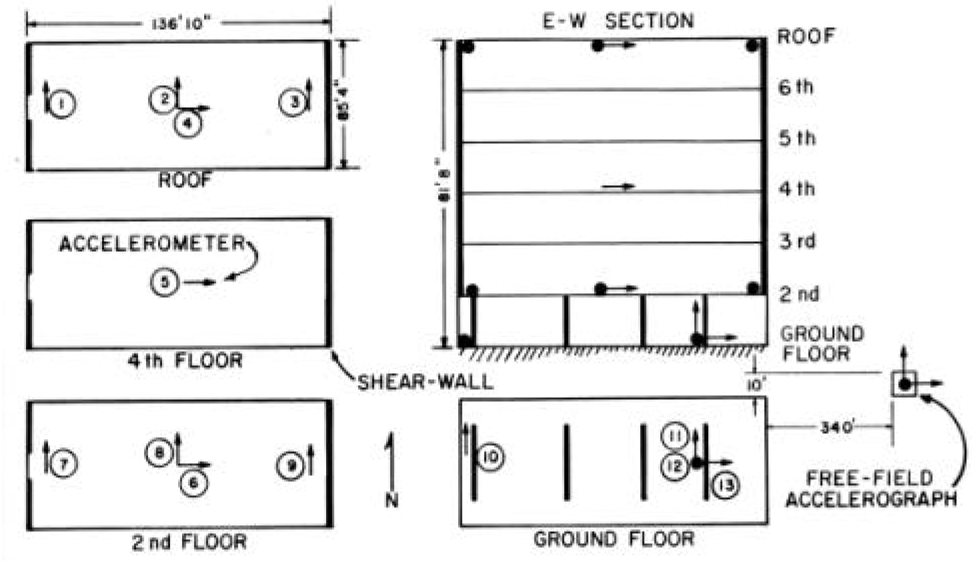

Figure 4. Locations of force-balance accelerometers (arrows with numbers) and SMA-1 accelerograph at Imperial County Service Building and adjacent free-field site (after Rojahn and Ragsdale, 1980). Arrows denote direction of positive acceleration on trace.

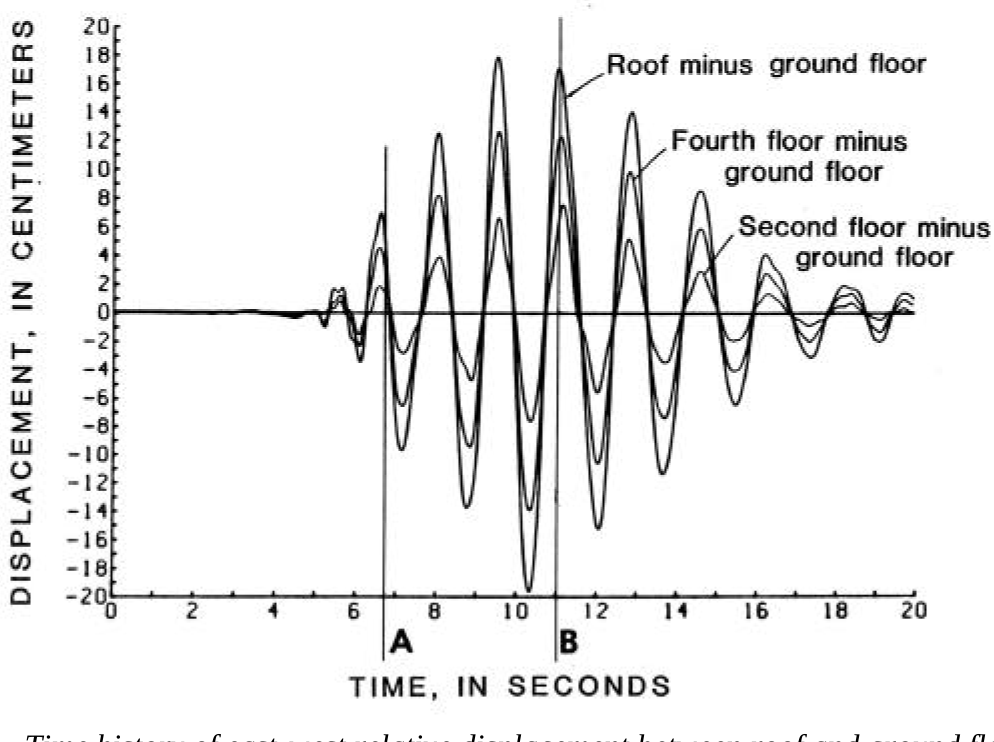

Figure 5. Time history of east-west relative displacement between roof and ground floor (difference of data from accelerometers 4 and 13), between fourth and ground floors (difference of data from accelerometers 5 and 13), and between second and ground floors (difference of data from accelerometers 6 and 13). A and B are interpreted to be the times at which damage initiated and the columns collapsed at the east side of the building (see Figure 3).

Concluding Remarks

This paper briefly describes that portion of the ATC-54 *Guidelines for Using Strong-Motion Data for Postearthquake Response and Postearthquake Structural Evaluation* (currently under development by the Applied Technology Council), pertaining to rapid evaluation of structures using strong-motion data. The implicit definition of "rapid" is hours to days after a damaging earthquake occurs, as opposed to instantly, or in real time. Recommended procedures include methods for (1) determining the extent to which a structure's natural period of vibration has lengthened, either permanently or temporarily (if permanent, a loss in stiffness is implied), (2) evaluating recorded motions and comparing them to seismic design loading criteria, such as effective peak acceleration or acceleration response ordinates; and (3) estimation of roof drift and inter-story drift. The methods have been taken from the existing literature and no research was performed to develop new methods.

The ATC-54 *Guidelines* also provide information on sources of strong-motion data, and guidance on the limitations of strong-motion data, as related to rapid evaluation for determination of possible earthquake-induced structural damage.

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IMPROVED DAMAGE PARAMETERS FOR POST-EARTHQUAKE APPLICATIONS

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Abstract

Various ground shaking and damage parameters are examined for post-earthquake applications. Peak ground motion values, elastic response spectra, spectrum intensity, drift spectrum, inelastic spectra, hysteretic energy spectrum, and two improved damage spectra are examined. The proposed damage spectra will be zero if the response remains elastic, and will be unity when the displacement capacity under monotonic deformation is reached. The damage spectra can be reduced to the special cases of normalized hysteretic energy and displacement ductility spectra. The proposed damage spectra are promising for seismic vulnerability studies and post-earthquake applications of existing facilities, and performance-based design of new structures.

Introduction

The objectives of this study are to examine various existing ground shaking, response and damage parameters and also to develop an improved damage parameter for post-earthquake applications. There are numerous ground shaking and damage parameters available. These include: peak ground acceleration, peak ground velocity, elastic response spectra, spectrum intensity, inelastic response spectra, interstory drift ratio, drift spectrum, and hysteretic energy spectra, among others.

In this study the above parameters are examined. Additionally, improved *damage spectra* are introduced and examined in details. The damage spectra are based on normalized response quantities of a series of inelastic single-degree-of-freedom (SDOF) systems. The proposed damage spectra will be zero if the structure remains elastic, and will be unity under the extreme condition of reaching the maximum deformation capacity under monotonically increasing lateral deformation. Following an earthquake, generation of near-real time contour maps of damage spectral ordinates can provide information on the spatial distribution of damage potential of the recorded ground motions for specified types of structures. Such maps can be useful for various post-earthquake applications, damage assessments, and emergency response; as well as for evaluation of the damage potential of earthquakes. Utilization of an up-to-date inventory of existing structures enhances the reliability of such maps in identifying the damaged areas.

Various ground shaking parameters as well as the proposed damage spectra are computed for hundreds of the ground motions recorded during the Northridge and Landers earthquakes. Additionally, these parameters are compared for specific cases of a seven-story reinforced concrete (RC) frame, and 17 low-rise ductile RC frames affected by the Northridge earthquake.

Shaking and Damage Parameters Considered

Following an earthquake, maps of the spatial distribution of the recorded and computed data are rapidly generated and posted on the Internet by TriNet (Wald, et al., 1999). These maps are used for a wide variety of post-earthquake applications. Currently six maps are generated: contour maps of peak ground acceleration (PGA), peak ground velocity (PGV), elastic spectral accelerations at periods 0.3, 1.0, and 3.0 seconds, and instrumentally derived seismic intensity. In this study the following ground shaking, response and damage parameters are also examined.

Damage Spectrum: Structural performance or damage limit states can be quantified by *damage indices* (DIs). A damage index is a normalized quantity that will be zero if the structure remains elastic (i.e., no significant damage is expected), and will be one if there is a potential of structural collapse. Other structural performance states (such as minor, moderate and major damages) fall between zero and one.

Damage spectrum represents variation of a damage index versus structural period for a series of SDOF systems subjected to a recorded ground motion. Bozorgnia and Bertero (2001a, b) introduced *two improved DIs and their corresponding damage spectra to quantify damage potential of the recorded earthquake ground motions*. The improved damage spectra explicitly satisfy the structural performance definitions at the limit states of being zero and one. Details of the definitions and characteristics of these damage spectra are presented in the following section. Damage spectra for hundreds of horizontal accelerations recorded during the Northridge and Landers earthquakes are computed, and to demonstrate an application of such spectra, contour maps of damage spectral ordinates are plotted.

Displacement Ductility: Structural damage is usually associated with inelastic response rather than elastic structural behavior. *Displacement ductility*, μ, defined as the maximum displacement of an inelastic SDOF system divided by the yield displacement, is a measure of inelastic response. Ductility spectrum, which is the variation of μ with period, can provide some useful information about general inelastic response behavior. Characteristics of inelastic spectra and the contrasts between inelastic and elastic spectra have been extensively studied for various input ground motions (e.g., Newmark and Hall, 1982; Bertero, et al., 1978; Mahin and Bertero, 1981, among other studies). Ductility spectra for hundreds of horizontal ground accelerations recorded during the Northridge and Landers, California, earthquakes are computed for 20 structural periods ranging from 0.1 to 4.0 seconds, and contours of constant ductility are presented and examined.

Interstory Drift Ratio, or more properly ***Story Drift Ratio*** is the ratio of the maximum story displacement over the story height. It has both practical and experimental significance as a measure of structural and non-structural damage. For example, for the purpose of *performance–based seismic design*, "SEAOC Blue Book" (SEAOC, 1999) has provided tentative values for drift ratios associated with different structural performance states. The interstory drift ratios demanded by the recorded ground motions are estimated using the calculated maximum displacement.

Hysteretic Energy: E_H, is a measure of the inelastic energy dissipation demanded by the earthquake ground motion (Mahin and Bertero, 1981; Uang and Bertero, 1990; Bertero and Uang, 1992). Hysteretic energy includes cumulative effects of repeated cycles of inelastic response and, therefore, the effects of strong-motion duration are included in this quantity. If the response of the structure remains elastic, E_H will be zero. “Equivalent hysteretic energy velocity,” $V_H = (2\ E_H/M)^{1/2}$ has also been used (Uang and Bertero, 1988), where M is the mass of the SDOF system.

Housner Spectrum Intensity: Housner (1952) defined spectrum intensity (SI) as the area under the pseudo-velocity response spectrum over a period range of 0.1 to 2.5 seconds. It is a measure of the intensity of ground shaking for elastic structures (Housner, 1975). SI is computed for 5% damping for hundreds of horizontal ground acceleration records. Contour map of SI for the Northridge earthquake is also presented.

Drift Spectrum: This quantity represents maximum story drift ratio in multi-story buildings demanded by the ground motion (Iwan, 1997). The formulation is based on linear elastic response of a uniform continuous shear-beam model. It requires ground velocity and displacement histories as input motions.

There are other shaking parameters whose characteristics and effects directly or indirectly are included in the above parameters. For example, a parameter of interest is the duration of strong ground motion (Bolt, 1973; Trifunac and Brady, 1975). The effects of the strong-motion duration through repeated cycles of inelastic response are included in the hysteretic energy and damage spectra.

Another parameter of interest is Arias Intensity (Arias, 1970), which, in its commonly used version, is the area under the total energy spectrum in an undamped elastic SDOF system. Energy in SDOF systems is included in both V_H and damage spectra; and in this study, these parameters are evaluated over a wide range of natural periods.

Damage Spectra

In the following section a brief overview of various damage indices is provided. Improved damage indices are then introduced and damage spectra are presented.

Review of Most Commonly Used Damage Indices

A damage index (DI) is based on a single or combination of structural response parameters such as force, deformation and energy dissipation. One method of computing the DI is to compare the response parameters demanded by the earthquake with the structural “capacities” (Powell and Allahabadi, 1988). Traditionally, the “capacities” or ultimate values of the response parameters are defined in terms of their maximum values under monotonically increasing deformations. For example, a fraction of the ultimate deformation capacity of the system under monotonically increasing lateral deformation (u_{mon}) has been used as the deformation capacity during the earthquake motion.

There are different damage indices available. For example, damage index may be based on plastic deformation (e.g., Powell and Allahabadi, 1988; Cosenza, et al., 1993):

$$DI_\mu = (u_{max}-u_y)/(u_{mon}-u_y) = (\mu - 1) / (\mu_{mon} - 1) \qquad (1)$$

where u_{max} and u_y are the maximum and yield deformations, respectively, and u_{mon} is maximum deformation capacity of the system under a monotonically increasing lateral deformation. In equation (1) $\mu = u_{max}/u_y$ is displacement ductility demanded by the earthquake and $\mu_{mon} = u_{mon}/u_y$ is "monotonic ductility capacity".

Displacement ductility alone does not reveal information on the repeated cycles of inelastic deformations and energy dissipation demand (e.g., Mahin and Bertero, 1981; Mahin and Lin, 1983). Hence, other structural response parameters such as hysteretic energy dissipation has also been used. Seismic input energy to a structural system (E_I) is balanced by (Uang and Bertero, 1988; and 1990):

$$E_I = E_H + E_K + E_S + E_\xi \qquad (2)$$

where E_H , E_K , E_S and E_ξ are irrecoverable hysteretic energy, kinetic energy, recoverable elastic strain energy, and viscous damping energy, respectively. Hysteretic energy (E_H) includes cumulative effects of repeated cycles of inelastic response and is usually associated with the structural damage. If the response of the structure remains elastic, E_H will be zero, by its definition. For SDOF systems, Mahin and Bertero (1976; and 1981) defined normalized hysteretic energy $E_H/(F_y\, u_y)$ and its corresponding normalized hysteretic energy ductility:

$$\mu_H = E_H/(F_y\, u_y) + 1 \qquad (3)$$

where F_y and u_y are yield strength and deformation of the system, respectively. Numerically μ_H is equal to the displacement ductility of a monotonically deformed equivalent elastic-perfectly-plastic (EPP) system that dissipates the same hysteretic energy, and has the same yield strength and initial stiffness as the actual system.

A damage index can be based on hysteretic energy. For example, for EPP systems, Cosenza, et al. (1993) and Fajfar (1992) used:

$$DI_H = [E_H/(F_y\, u_y)] \,/\, (\mu_{mon} - 1) \;=\; (\mu_H - 1) \,/\, (\mu_{mon} - 1) \qquad (4a)$$

For a general force-deformation relationship, the above DI can be rewritten (Cosenza, et. al,1993):

$$DI_H = E_H \,/\, E_{Hmon} \qquad (4b)$$

where E_{Hmon} is hysteretic energy capacity of the system under monotonically increasing deformation.

A combination of maximum deformation response and hysteretic energy dissipation was proposed by Park and Ang (1985):

$$DI_{PA} = (u_{max} \,/\, u_{mon}) + \beta\, E_H/(F_y\, u_{mon}) \qquad (5)$$

where $\beta \geq 0$ is a constant, which depends on structural characteristics. DI_{PA} has been calibrated against numerous experimental results and field observations in earthquakes (e.g., Park et al., 1987; Ang and de Leon, 1994). $DI_{PA} < 0.4$ to 0.5 has been reported as the limit of repairable damage (Ang and de Leon, 1994). Cosenza, et al. (1993) reported that experimental-based values of β have a median of 0.15 and for this value, DI_{PA} correlates well with the results of other damage models proposed by Banon and Veneziano (1982) and Krawinkler and Zohrei (1983). DI_{PA} has drawbacks; two of them will be mentioned here. First, for elastic response, when E_H=0 and the damage index is supposed to be zero, the value of DI_{PA} will be greater than zero. The second disadvantage of DI_{PA} is that it does not give the correct result when the system is under monotonic deformation. Under such a deformation, if the maximum deformation capacity (u_{mon}) is reached, the value of the damage index is supposed to be 1.0, i.e., an indication of potential of failure. However, as it is evident from (5), DI_{PA} results in a value greater than 1.0. Chai et al. (1995) modified DI_{PA} to correct the second deficiency of DI_{PA}, as mentioned above; however, the first deficiency of DI_{PA} was not corrected. Despite its drawbacks, DI_{PA} has been extensively used for different applications. This is, in part, due to its simplicity and its extensive calibration against experimentally observed seismic structural damage.

Improved Damage Indices

Bozorgnia and Bertero (2001a, b) introduced two improved damage indices for a generic inelastic SDOF system. These damage indices are as follows:

$$DI_1 = [(1 - \alpha_1)(\mu - \mu_e) / (\mu_{mon} - 1)] + \alpha_1 (E_H / E_{Hmon}) \quad (6)$$

$$DI_2 = [(1 - \alpha_2)(\mu - \mu_e) / (\mu_{mon} - 1)] + \alpha_2 (E_H / E_{Hmon})^{1/2} \quad (7)$$

where,

$$\mu = u_{max} / u_y = \text{Displacement ductility} \quad (8a)$$

$$\mu_e = u_{elastic} / u_y = \text{Maximum elastic portion of deformation} / u_y \quad (8b)$$

= 1 for inelastic behavior; and

= μ if the response remains elastic

μ_{mon} is monotonic displacement ductility capacity, E_H is hysteretic energy demanded by the earthquake ground motion, E_{Hmon} is hysteretic energy capacity under monotonically increasing lateral deformation, and $0 \le \alpha_1 \le 1$ and $0 \le \alpha_2 \le 1$ are constants. Using the definition of hysteretic ductility μ_H (Mahin and Bertero,1976; and 1981) given in equation (3) for both earthquake and monotonic deformations, the new damage indices can be rewritten as:

$$DI_1 = [(1 - \alpha_1)(\mu - \mu_e) / (\mu_{mon} - 1)] + \alpha_1 (\mu_H - 1) / (\mu_{Hmon} - 1) \quad (9)$$

$$DI_2 = [(1 - \alpha_2)(\mu - \mu_e) / (\mu_{mon} - 1)] + \alpha_2 [(\mu_H - 1) / (\mu_{Hmon} - 1)]^{1/2} \quad (10)$$

For the special case of elastic-perfectly-plastic (EPP) systems:

$$E_{Hmon} = F_y (u_{mon} - u_y) \text{ and } \mu_{Hmon} = \mu_{mon} \quad (11)$$

$$DI_1 = [(1 - \alpha_1)(\mu - \mu_e) / (\mu_{mon} - 1)] + \alpha_1 (E_H / F_y u_y) / (\mu_{mon} - 1) \quad (12)$$

$$DI_2 = [(1 - \alpha_2)(\mu - \mu_e) / (\mu_{mon} - 1)] + \alpha_2 [(E_H / F_y u_y) / (\mu_{mon} - 1)]^{1/2} \quad (13)$$

Few characteristics of the improved damage indices are listed below:

1) If the response remains elastic, i.e., when there is no significant damage, then $\mu_e = \mu$ and $E_H = 0$, and consequently both DI_1 and DI_2 will become zero. This is a characteristic expected for any damage index.

2) Under monotonic lateral deformation if $u_{max} = u_{mon}$, the damage indices will be unity. This is true for a general force-deformation relationship.

3) If $\alpha_1 = 0$ and $\alpha_2 = 0$, damage indices DI_1 and DI_2 (equations 6 and 7) will be reduced to a special form given in equation (1). In this special case, the damage index is assumed to be only related to the maximum *plastic* deformation.

4) If $\alpha_1 = 1$ and $\alpha_2 = 1$, damage indices DI_1 and DI_2 will be only related to the hysteretic energy dissipation E_H. Specifically, in this case, damage index DI_1 will be reduced to a special form given in equation (4b). If additionally the force-deformation relationship is EPP, damage index DI_1 given in (12) will be reduced to a special form given in equation (4a).

5) Equivalent hysteretic velocity V_H (Uang and Bertero, 1988) was defined as:

$$V_H = (2\ E_H/M)^{1/2} \tag{14}$$

where M is the mass of the system. It is evident from the definition of DI_2 given in (7) that DI_2 is related to the normalized equivalent hysteretic velocity. If V_H spectrum is already available, DI_2 can be easily generated.

Development of Damage Spectra

As mentioned before, damage spectrum of a recorded ground motion represents variation of a damage index versus structural period for a series of SDOF systems. Once a damage index, such as DI_1 and DI_2, is defined, damage spectrum can be constructed. The steps involved in developing the damage spectra are summarized in Figure 1.

Examples of damage spectra are presented in Figure 2. This figure shows damage spectra for the 1940 Imperial Valley earthquake recorded at El Centro, and for the Northridge earthquake recorded at Sylmar County Hospital. For this figure, the following characteristics were used: viscous damping ξ=5%; EPP force-displacement relationship; yield strength was based on elastic spectrum of UBC-97 (without near-source factors) reduced by a factor of 3.4; also μ_{mon}=10, α_1=0.269, α_2=0.302 were used. These values for α_1 and α_2 are based on an analysis of the Northridge earthquake records, as explained below. Computer program Nonspec (Mahin and Lin, 1983) was employed to compute the basic response parameters such as displacement ductility and hysteretic energy demands. DI_1 and DI_2 were then computed according to equations (12) and (13). The damage spectra for periods longer than 0.5 sec are plotted in Figure 2. For the structures with shorter periods, generally larger over-strength factor and μ_{mon} should be used. The contrast between the two damage spectra presented in Figure 2 is an evidence of very different damage potentials of these two ground motion records for the SDOF systems considered.

As mentioned previously, DI_{PA} has been already calibrated against numerous experimental and field cases. However, because of its deficiencies, it is not reliable at its low and high values. Thus, in the intermediate range of the damage index, a comparison between values of DI_1 with those of DI_{PA} can result in an estimate for α_1. Hence, the following procedure was used to estimate α_1: ductility and hysteretic energy spectra and DI_{PA} were computed at 20 structural periods ranging from 0.1 to 4.0 seconds using 220 horizontal ground acceleration records of the Northridge earthquake. Then coefficient α_1 was determined through regression analyses, i.e., by comparing values of DI_1 with those of DI_{PA} (for $0.2<DI_{PA}<0.8$). A similar process was repeated to estimate coefficient α_2 in DI_2. The same procedure was also carried out using 176 horizontal acceleration records of the 1992 Landers, California, earthquake. The computed α_1 and α_2 coefficients using the ground motion records of the Northridge and Landers earthquakes are listed in Table 1. Subsets of the results of the regression analyses for the Northridge earthquake are also graphically presented in Figure 3.

Effects of Strong-Motion Duration

Experimental studies have demonstrated that failures of structural members and systems are influenced by the number of inelastic cycles of response (e.g., Bertero, et al., 1977). In other words, structural systems generally become more vulnerable if they go through repeated cycles of inelastic motions. This generally occurs when the structure is subjected to a long duration damaging ground motion. Hence, in quantifying damage potential of the recorded ground motion it is desirable to include the effects of strong ground motion duration.

Hysteretic energy E_H through its definition (e.g., Uang and Bertero, 1981) is a cumulative quantity. More cycles of inelastic deformations correspond to a larger value for the hysteretic energy dissipation. Thus, the effects of repeated cycles of inelastic response and strong-motion duration are reflected in E_H. Hence, in the damage indices that include hysteretic energy terms, the effects of repeated cycles of inelastic deformations and strong-motion duration are also included. An example of the duration effect on E_H and damage spectrum is shown in Figure 4.

In 1999 two major earthquakes occurred in Turkey: (1) on August 17, 1999 an earthquake of magnitude Ms 7.8; and (2) on November 12, 1999 another major earthquake of magnitude 7.5 (EERI, 2000). For both events, the ground accelerations were recorded at Duzce station. Figure 4 shows the ground accelerations recorded in these two events, with 10 seconds of zero ground acceleration added in between. Time variation of the hysteretic energy demand is also plotted in Figure 4. Damage spectra of the first and second events individually, as well as the damage spectrum of the combined acceleration records were computed and presented in Figure 4. The results shown in this figure are based on the same basic parameters as used in Figure 2 (except, $\mu_{mon}=8$, $\alpha_1=0.286$, and including near-source factors). Displacement ductility spectra are also plotted in Figure 4. As it is expected, the time variation of the hysteretic energy clearly shows that E_H incorporates the cumulative effects due to the strong-motion duration. Because the damage spectrum includes E_H spectrum (see Figure 1), the damage spectrum is also influenced by the cumulative effects. Such a cumulative energy effect, however, is not included in the displacement ductility spectra. It should be noted that the effects of the sequence, and therefore the history, of different hysteretic loops are not considered in the proposed damage spectra.

Attenuation of Damage Spectra

Once damage spectral ordinates for numerous ground motion records are computed, it is possible to evaluate the attenuation of damage spectra. Such an attenuation model can be used to estimate the variation of the damage spectral ordinates with site-to-source distance. To demonstrate the concept, attenuation of the damage spectral ordinates for the Northridge earthquake was computed. First, damage spectra for the horizontal accelerations recorded at alluvial sites during the Northridge earthquake were calculated for the same set of parameters as used for Figure 2. Then, regression analyses were performed on the following attenuation model:

$$\ln (DI_1) = a + d \ln [R^2 + c^2]^{1/2} + e \qquad (15)$$

where R is the closet distance from the site to the surface projection of the fault plane, e is a random error, and a, c, and d are the regression parameters to be computed. Site soil conditions at the recording stations and site-to-source distances were taken from a comprehensive ground motion database compiled by Campbell and Bozorgnia (2000) and Bozorgnia et al. (1999). The distance scaling of the damage spectral ordinates is shown in Figure 5. The median damage spectra for distances 3, 10, 20, and 40 km form the fault are also plotted in the same figure. It should be noted that the damage spectra shown in Figure 5 are based on the assumption that structural over-strength factor and μ_{mon} are constant over the period range. These factors, however, are possibly higher at short periods (e.g., for low-rise buildings) than those at long periods.

Spatial Distribution of Various Parameters

For any specified structural characteristics and using the recorded ground motions at various recording stations, it is possible to rapidly generate damage spectra and plot their spatial distribution at selected periods. As an example, contours of damage spectral ordinates based on 220 horizontal accelerations recorded during the Northridge earthquake are plotted in Figure 6 for periods 1.0 and 3.0 seconds. For computation of damage spectra, the same basic parameters as Figure 2 were used, except μ_{mon} =12. Also, uniform basic structural characteristics over the area were assumed. Soil conditions at the recording stations were taken from the strong-motion database compiled by Campbell and Bozorgnia (2000) and Bozorgnia et al. (1999). The soil conditions were used to adjust F_y/W of the SDOF system at the recording site (see Figure 1). At each recording station the maximum of the damage spectral ordinates for the two horizontal components was taken. Figure 6 also shows the epicenter of the earthquake and the surface projection of the fault plane. Contour plots for DI_2 (not shown here) are very comparable to those plotted in Figure 6. As mentioned above, this figure is for uniformly distributed structural characteristics in the area, except for the adjustment of F_y/W for the local soil conditions. However, the distribution of the damage spectral ordinates can be modified to incorporate the data from an inventory of the existing structures in the area. For example, for buildings, data on the structural material, structural system, number of stories, age of the structure, etc. can be approximately translated into the basic structural data needed to generate damage spectra. If a better estimate of the spatial distribution of the basic structural characteristics is used, more realistic contour plots of the damage spectral ordinates can be generated.

Plotted in Figure 6 are also the distributions of the displacement ductility demanded by the recorded ground motion at periods 1.0 and 3.0 seconds. The same basic parameters were used as those for the damage spectral ordinates.

Given the displacement ductility, and consequently the maximum displacement of the SDOF system, interstrory drift ratio can be estimated. The following procedure was implemented: First, given the specified structural period, building height was estimated using the period-height relationship suggested by Goel and Chopra (1997) for reinforced concrete moment-resisting frames. For the purpose of estimating the drift, the smaller height estimated by the period formulas, was used. Then, the tentative guidelines provided by SEAOC (1999), Appendix I, were used to approximately estimate the interstory drift ratio. Figure 7 shows an example of the contour plots of interstory drift demanded by the recorded ground motion.

Housner spectrum intensity (Housner, 1952; and 1975) for 5% damping was also computed for the horizontal accelerations recorded in the Northridge earthquake. At each recording station, maximum of the spectrum intensities of the two horizontal components was taken. Spatial distribution of the spectrum intensity is also shown in Figure 7.

Selected results for the Landers earthquake are shown in Figure 8. This figure shows the damage spectral ordinates DI_1, with the same SDOF characteristics as used in Figure 2, except for μ_{mon}=12, and α_1=0.316. Again, a uniform distribution of structural characteristics in the area was used. The fault trace and the epicenter of the earthquake are also mapped in the figure. Displacement ductility and interstory drift ratio at period 1.0 second are also presented in this figure. For the computation of the interstory drift ratio, the same procedure was used as that for Figure 7.

Various contour plots in Figures 6-8 reveal information about different measures of the severity of the recorded ground motion at different locations. Some of them also reveal more information about performance of a set of simple structural models subjected to the recorded ground motion. One obvious advantage of the spectral damage contour plots is that they conveniently represent normalized values. For example, compare the contour plots of the damage spectral ordinates with displacement ductility demands. In order to compare them, ranges of ductility values need to be correlated to various structural performance descriptions (such as “operational”, “life safe”, “near collapse”, etc.). Using, for example, such a tentative correlation given in SEAOC (1999), Appendix I, contour plots of the damage spectra and displacement ductility are generally consistent. However, the damage spectral plots, representing a normalized quantity, are more convenient for post-earthquake applications. Additionally, as mentioned before, they include more features of the inelastic response than most of the other parameters considered.

Comparison of Parameters for Specific Cases

Ground shaking and damage parameters considered were compared for specific cases. This section presents a summary of the comparison.

Van Nuys seven-story hotel is an instrumented building which experienced major structural damage during the Northridge earthquake (California Seismic Safety Commission, 1996; Moehle, et al., 1997). The structure is a seven-story reinforced concrete frame building constructed in 1966 and no seismic retrofit work was performed prior to the Northridge earthquake. The details of the building are reported by California Seismic Safety Commission (1996), and Moehle, et al. (1997). Structural damage was primarily restricted to the longitudinal perimeter frames, and the damage included column shear failures, and immediately after the earthquake the building was “red tagged”. Various ground motion and damage parameters were computed using the recorded accelerations at ground level in EW direction. For the damage spectrum, basic structural characteristics were taken from the previous detailed analyses (California Seismic Safety Commission, 1996; Moehle, et al., 1997). Figure 9 shows variation of damage index DI_1 versus μ_{mon}. Similar plot was also constructed for DI_2 (not shown here). Having assigned a damage index of 0.8 (for a "near collapse" damage state), value of μ_{mon} was estimated for different values of α_1 and α_2. Summary of the results are given in Table 2. A range of 4.2 to 5.6 was computed for μ_{mon}. Compared with the results of the previous detailed nonlinear analyses, this seems a reasonable range. Computed drift ratio of 1.3%, estimated by both inelastic SDOF and drift spectrum analyses, is also within the range of 1.2 to1.9% based on the recorded motions of the building. This example shows that it is possible to estimate the damage spectrum with a reasonable accuracy, if the needed basic structural characteristics are accurately estimated.

Another case study is a set of 17 low-rise (1 to 3-story) ductile moment-resisting reinforced concrete frame buildings constructed between 1979 and 1990. These buildings were affected by the Northridge earthquake. Singhal and Kiremidjian (1996) assigned damage indices (DIs) to these buildings. These assigned DIs were based on the reported repair costs, not rehabilitation costs, and not based on direct field observations. The assigned DIs are very low -- an indication that the damage to these buildings was not severe. The highest reported DI is 0.26 for building #17 located at about 1.8 km from the Rinaldi Receiving Station (RRS). For each building, the closest free-field recording station located at the same general soil category was identified in the present study. Using the recorded motions various shaking and damage parameters were computed. Regression analyses were performed to compare the computed and assigned values of the damage indices, and to estimate μ_{mon}. The results, although very scattered, all indicate that in order to obtain the very low assigned DIs, the values of μ_{mon} and/or over-strength factor should be very high. This is conceptually consistent with the general understanding that the available global ductility and over-strength factor for low-rise ductile buildings are high. Another observation is that the recorded motion at RRS, which is close to building #17, is a very strong motion by almost all measures. For example, the peak horizontal ground acceleration (PGA) is 0.84g, peak ground velocity (PGV) is 159 cm/sec (Iwan, 1997), elastic spectral ordinates between periods 0.3-0.5 sec exceed about 1.7g, spectrum intensity for 5% damping is 456cm, and drift spectrum for 5% damping at 0.3-0.5 sec is between 1.5 and 2.26%. However, as mentioned before, the assigned damage index is low (Singhal and Kiremidjian, 1996) indicating no significant damage.

Consider, for example, spectrum intensity at RRS (456 cm) and that at the Van Nuys seven-story building (174 and 230 cm, for the motion at the building base and free-field record, respectively). Purely based on this parameter, more damage may be expected at the site of building #17 than the Van Nuys seven-story building. However, this did not occur, because strength, deformation and energy dissipation capacities of the structures are important factors in controlling the response, and therefore the damage. These factors, however, do not have any influence on the computed values of, e.g., elastic spectral ordinates or spectrum intensity. Hence, such parameters alone cannot accurately predict the observed damage.

Strength Spectra for Constant Values of Damage Index

The results presented earlier in this paper were for existing structures. Such results can be used for seismic performance assessment of existing facilities. For performance-based design of new structures, the desired performance state and the corresponding value of the damage index (DI) are specified and the structural strength needs to be determined. Therefore, for new design, it is desirable to construct strength spectra for constant values of DI. Figure 10 shows an example of such strength spectra for the 1940 Imperial Valley earthquake, El Centro station (NS). In this figure, consistent with previous results, zero value for DI corresponds to elastic spectrum. Also, as expected, the design strength decreases by increasing the value of DI. In the lower range of DI, a moderate increase in DI (i.e., accepting minor damage) results in a significant reduction in the design strength. However, in the intermediate and upper ranges of DI, increasing DI does not result in a significant reduction in the design strength. Strength spectra for constant values of DI, such as those presented in Figure 10, are promising for preliminary performance-based design of new structures.

Concluding Remarks

In this study several ground shaking, response and damage parameters were computed and examined. Two improved damage spectra and their characteristics were also examined. The parameters considered in this study can be classified into the following categories:

- *Parameters that are purely measures of free-field ground motion*. These include PGA and PGV, which are amongst the most commonly used parameters measuring severity of the ground motion. However, they are independent of any data about the behavior of structural systems. Therefore, besides their other limitations, these parameters *alone* have limited capabilities to accurately predict damage.

- *Parameters that are related to the elastic response of SDOF and continuous beam models*. These include elastic spectral ordinates, spectrum intensity, and drift spectrum. Although these are also very important measures and their applications have been extensive, they do not include effects of inelastic structural response and repeated cycles of inelastic deformations, which are generally associated with damage.

- *Inelastic response spectra in the forms of displacement ductility, interstory drift ratio, and strength spectra*. These parameters reveal some fundamental features of inelastic response; however, the effects of number of cycles of inelastic response are not included.

- *Spectrum of hysteretic energy dissipation due to plastic deformations, and its associated equivalent hysteretic velocity spectra*. These parameters include some fundamental features of inelastic response as well as the effects of repeated cycles of inelastic deformations and strong ground motion duration. However, in order to use these parameters for rapid damage assessments and post-earthquake applications, they have to be compared with the energy dissipation capacity of the structure.

- *Damage spectrum*. It is based on normalized response quantities of a series of inelastic SDOF systems. The improved damage spectra presented here are based on promising damage indices (DIs). The proposed damage spectra explicitly satisfy two important conditions: They will be zero if the response remains elastic, i.e., no significant damage is expected; and will be unity when the maximum deformation capacity is reached under monotonically increasing lateral deformation. Larger damage spectral ordinates conceptually correspond to larger damage. Another characteristic of the proposed damage spectra is that by varying a coefficient (α_1 in DI_1, or α_2 in DI_2), they are reduced to the commonly used normalized hysteretic energy and displacement ductility spectra. Also, the damage spectra, in their general form, are influenced by the repeated cycles of inelastic deformations and strong-motion duration. Although the proposed

DIs may be further improved to include other features of inelastic response, they are more reliable indices than other commonly used DIs such as DI_{PA}.

The proposed damage spectra can be used to quantify the damage potential of the recorded ground motion and relate that to seismic structural performance categories. They are also effective quantities for post-earthquake applications and rapid identification of the damaged areas based on the recorded ground motions and the type of construction. Utilization of an up-to-date inventory of existing structures enhances the reliability of spatial distribution of the damage spectra. The damage spectra are also promising for performance-based design of new structures.

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Table 1—Results of Regression Analyses to Estimate a_1 and a_2

b (*)	m_{mon}	a_1 **Northridge EQ**	a_2 **Northridge EQ**	a_1 **Landers EQ**	a_2 **Landers EQ**
0.10	8	0.206	0.273	0.238	0.280
0.15	8	0.286	0.332	0.316	0.331
0.20	8	0.364	0.385	0.378	0.380
0.10	10	0.185	0.243	0.231	0.245
0.15	10	0.269	0.302	0.296	0.297
0.20	10	0.350	0.354	0.357	0.344

(*) See equation (5)

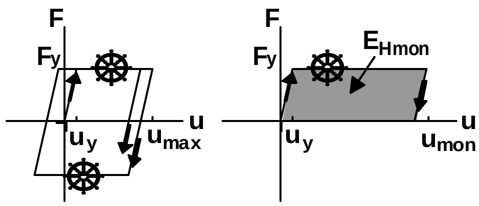
Table 2—Summary of the Recorded and Computed Data, Van Nuys Seven-Story Hotel, Northridge Earthquake

Peak Accel. (g)	**Elastic Spectrum (g) at 1.5 sec**	**Damage Spectrum at 1.5 sec**	m_{mon}	**Interstory Drift Ratio (%)(*)**	**Rel. Roof Disp. / Bldg Height (%) Based on Recorded Bldg Accelerations**	**Drift: 3rd-2nd Flrs (%) Based on Recorded Bldg Accelerations**	**Drift Spectrum at 1.5 sec (%)(**)**	**Spectrum Intensity (cm)(***)**
0.45	0.46	0.8	4.2 – 5.6	1.3	1.2	1.9	1.3	174

(*) Computed using SDOF response
(**) At Base level, for 5% damping
(***) Using the recorded accelerations at the ground level of the building (EW). From free-field contours: 230 cm

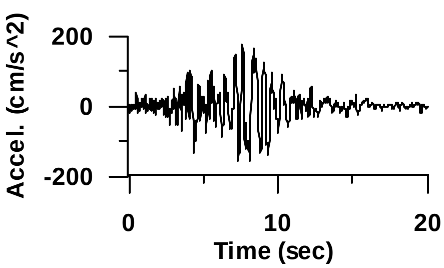
DAMAGE SPECTRA

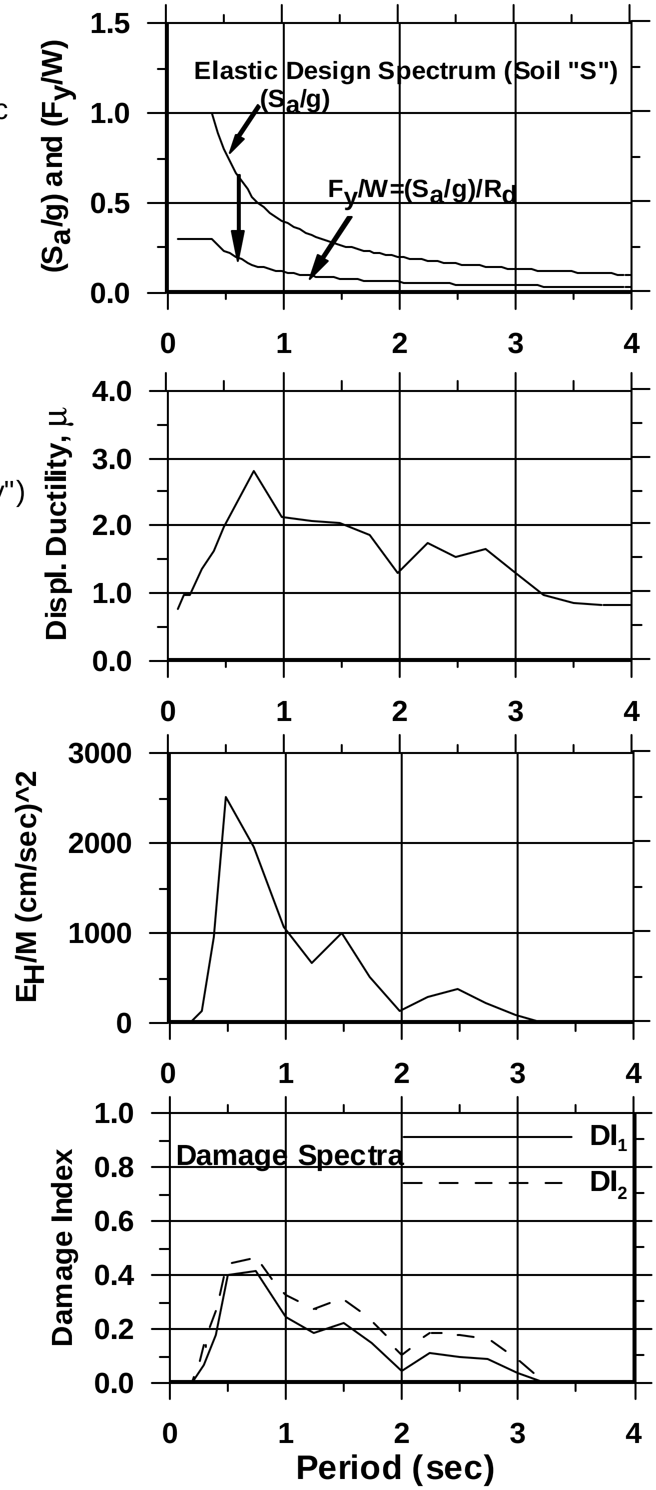
Given: A series of single-degree-of-freedom systems with:
- Period T (sec)
- Damping ratio ξ
- Site soil condition "S" (soil category)
- Strength; specified according to a seismic code: $(F_y/W) = (S_a/g)/R_d$
- A force-deformation relationship:



- E_{Hmon} ("monotonic hysteretic energy capacity") or, for elastic-perfectly-plastic (EPP): μ_{mon} ("monotonic ductility capacity")

Subjected to an earthquake ground acceleration record:





Compute:
- Ductility spectrum (μ)
- Hysteretic energy spectrum (E_H)

Develop Damage Spectra:
- By combining μ and E_H spectra:

$$DI_1 = [(1 - \alpha_1)(\mu - \mu_e)/(\mu_{mon} - 1)] + \alpha_1 (E_H/E_{Hmon})$$

$$DI_2 = [(1 - \alpha_2)(\mu - \mu_e)/(\mu_{mon} - 1)] + \alpha_2 (E_H/E_{Hmon})^{1/2}$$

where $0 \le \alpha_1 \le 1$ and $0 \le \alpha_2 \le 1$ are constants; μ_e = max. elastic deformation/u_y

Figure 1: Summary of steps involved in developing *Damage Spectra*

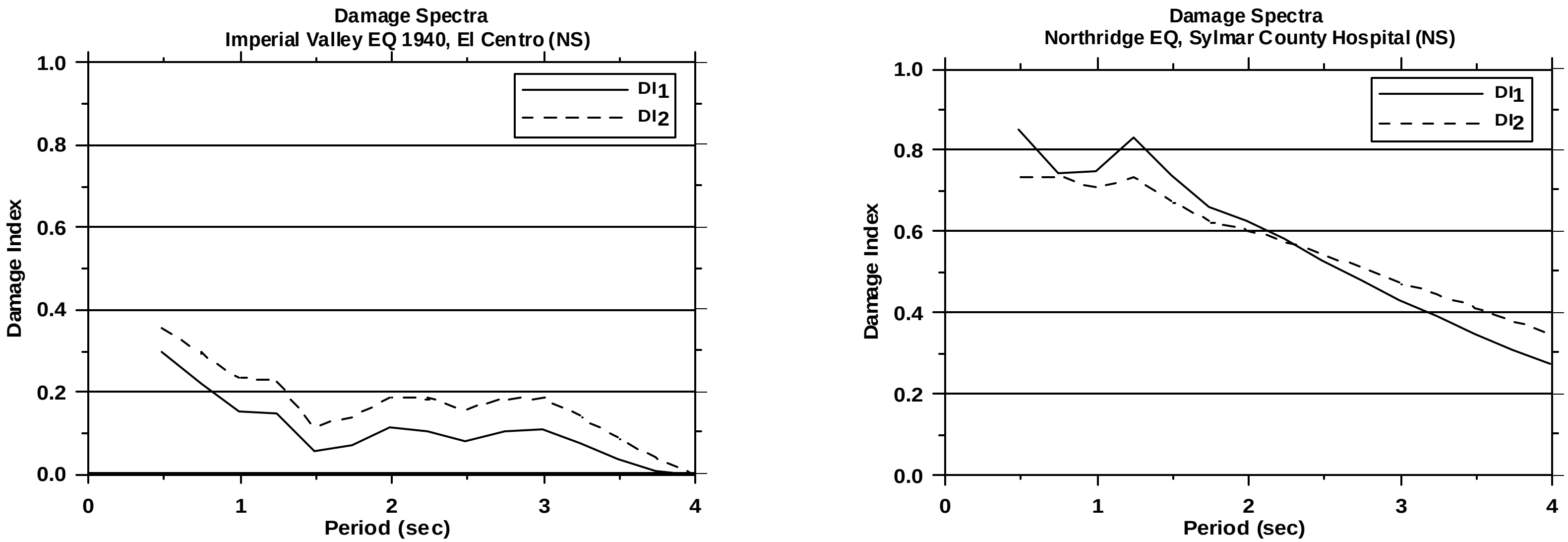


Figure 2: Examples of damage spectra, considering ξ=5%, μ_{mon}=10, and EPP behavior.

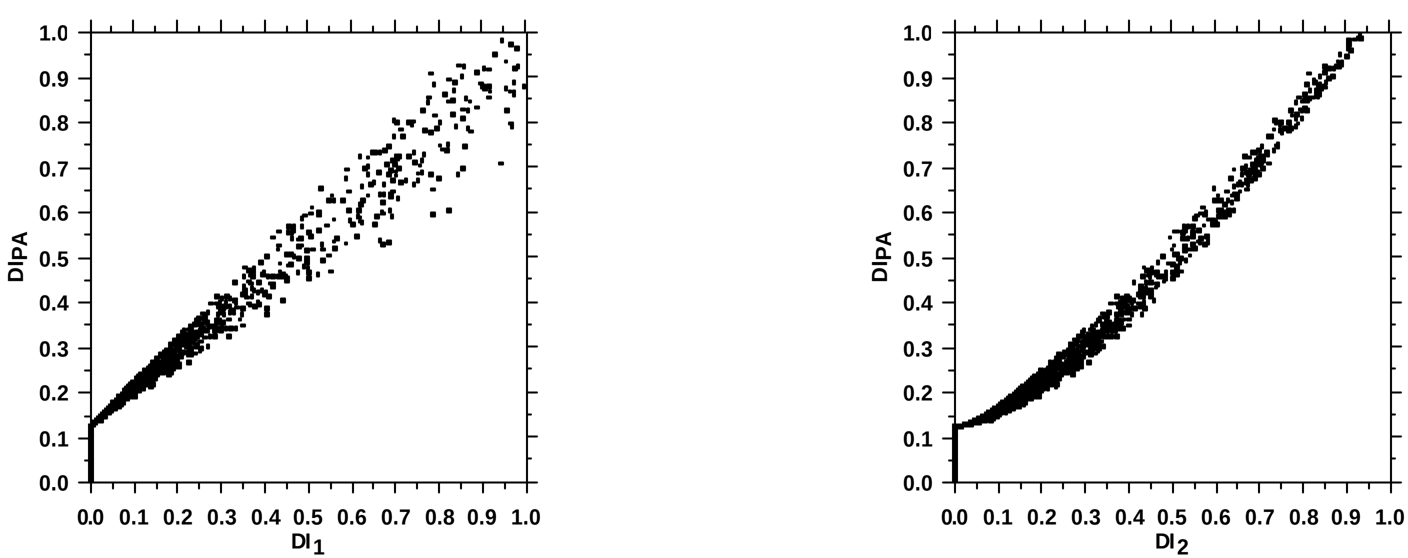


Figure 3: Example of correlation between damage indices (DI_1, DI_2) and DI_{PA}: Northridge EQ records, with β=0.15, α_1=0.286, α_2=0.332, μ_{mon}=8, ξ=5%.

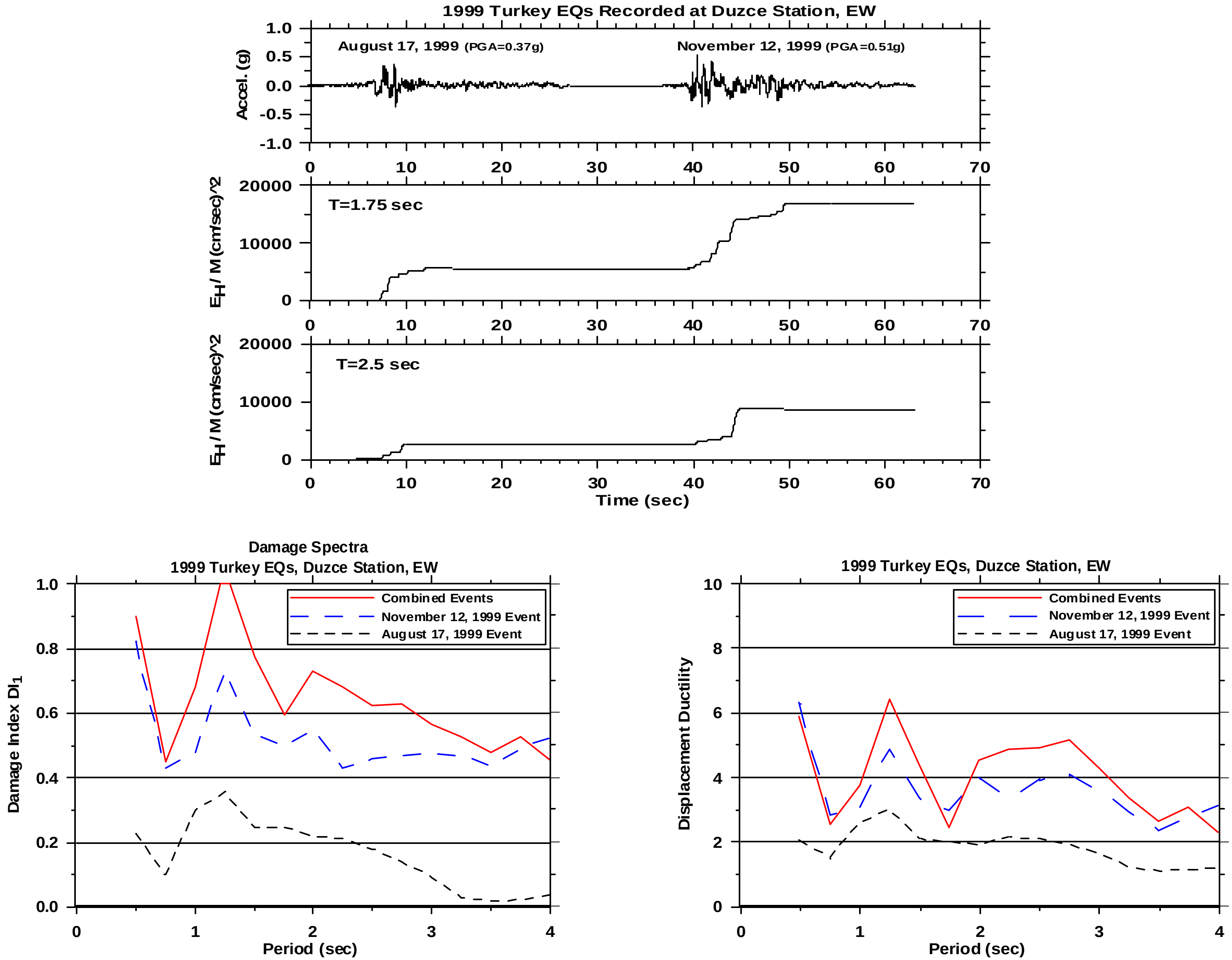


Figure 4: Ground accelerations at Duzce (Turkey); hysteretic energy demands; damage spectra; and displacement ductility.

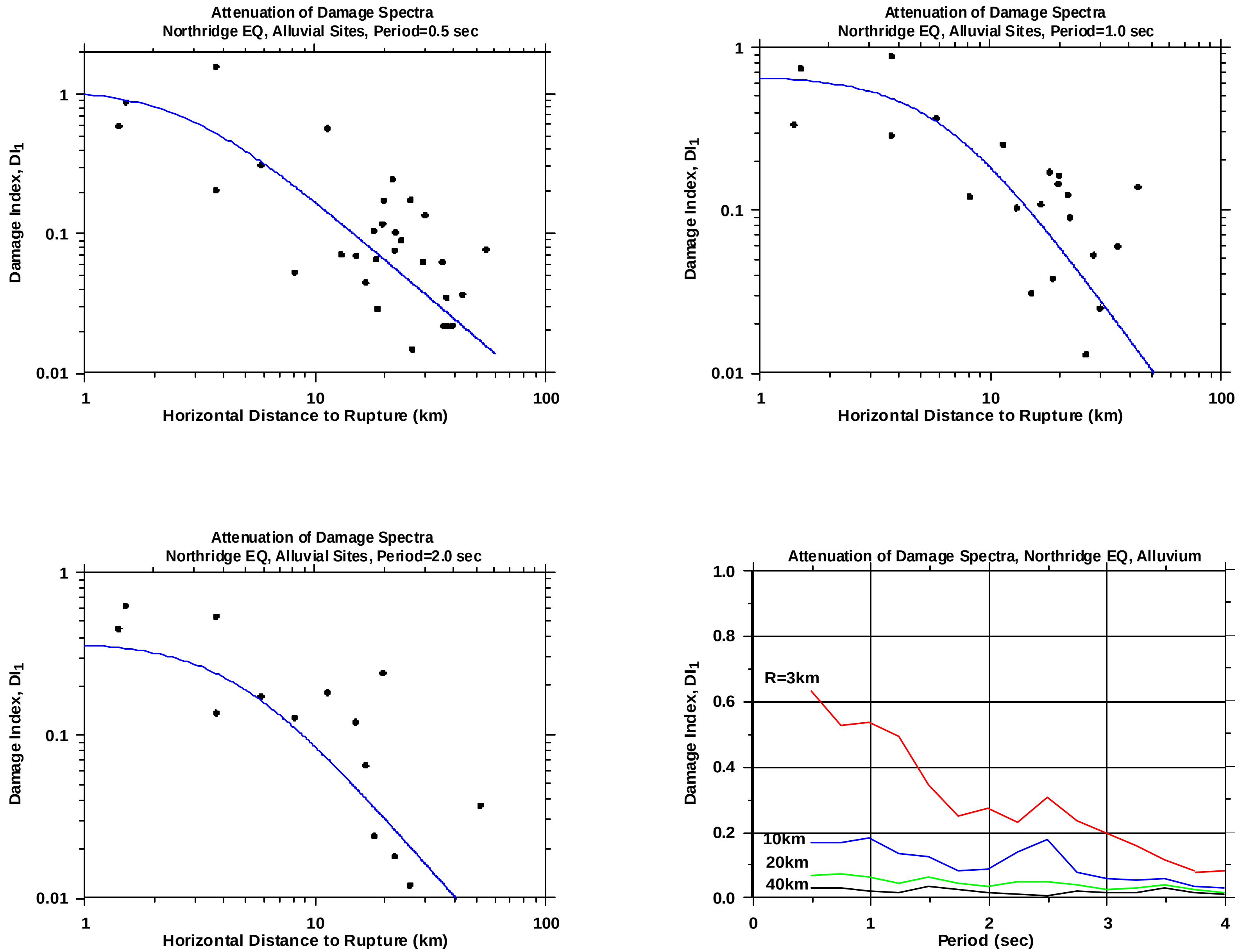


Figure 5: Distance scaling of damage spectral ordinates for the Northridge earthquake at alluvial sites.

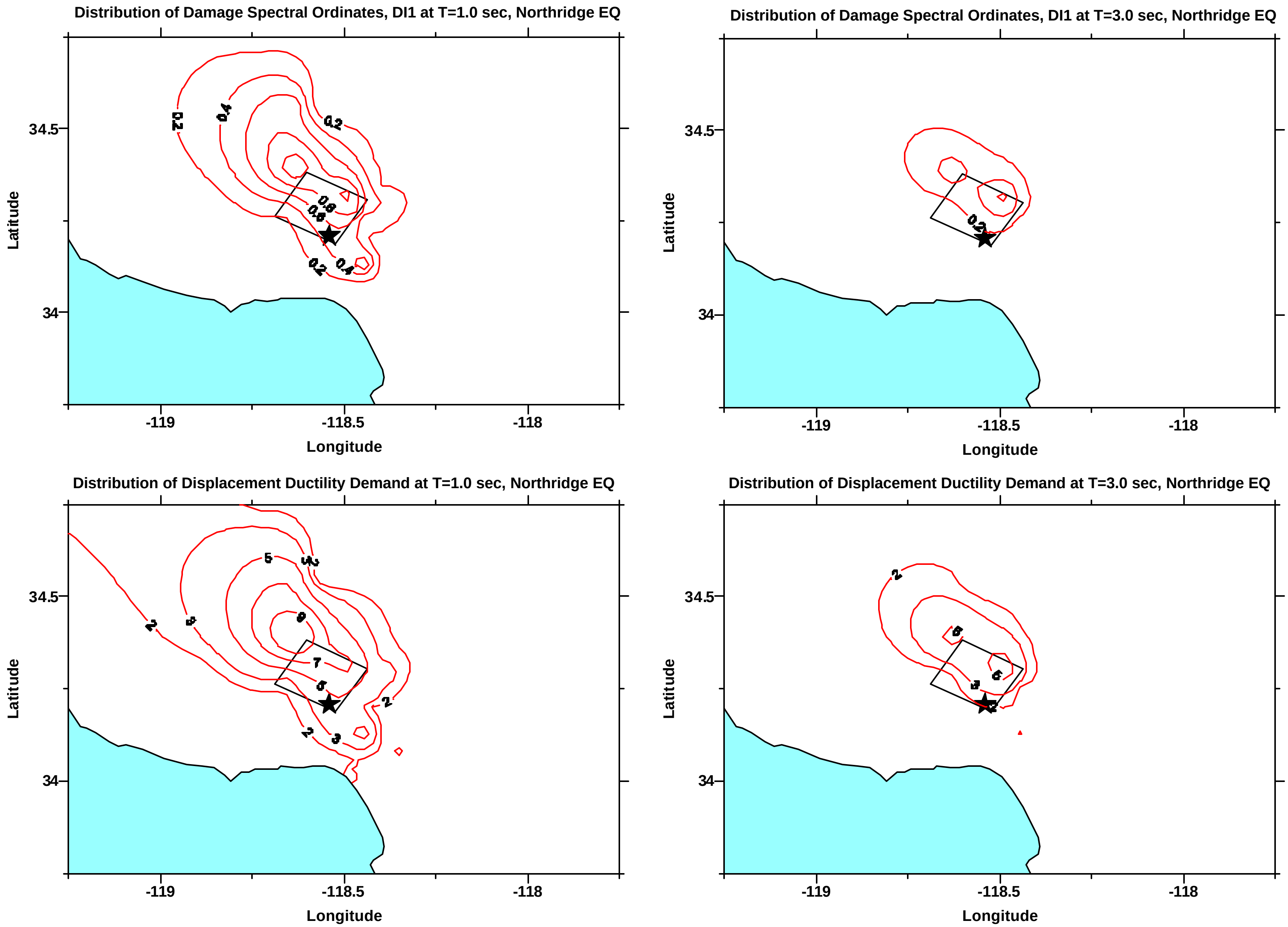


Figure 6: Distribution of damage spectral ordinates and displacement ductility demands of the ground motions recorded during the Northridge earthquake, T=1 and 3 sec.

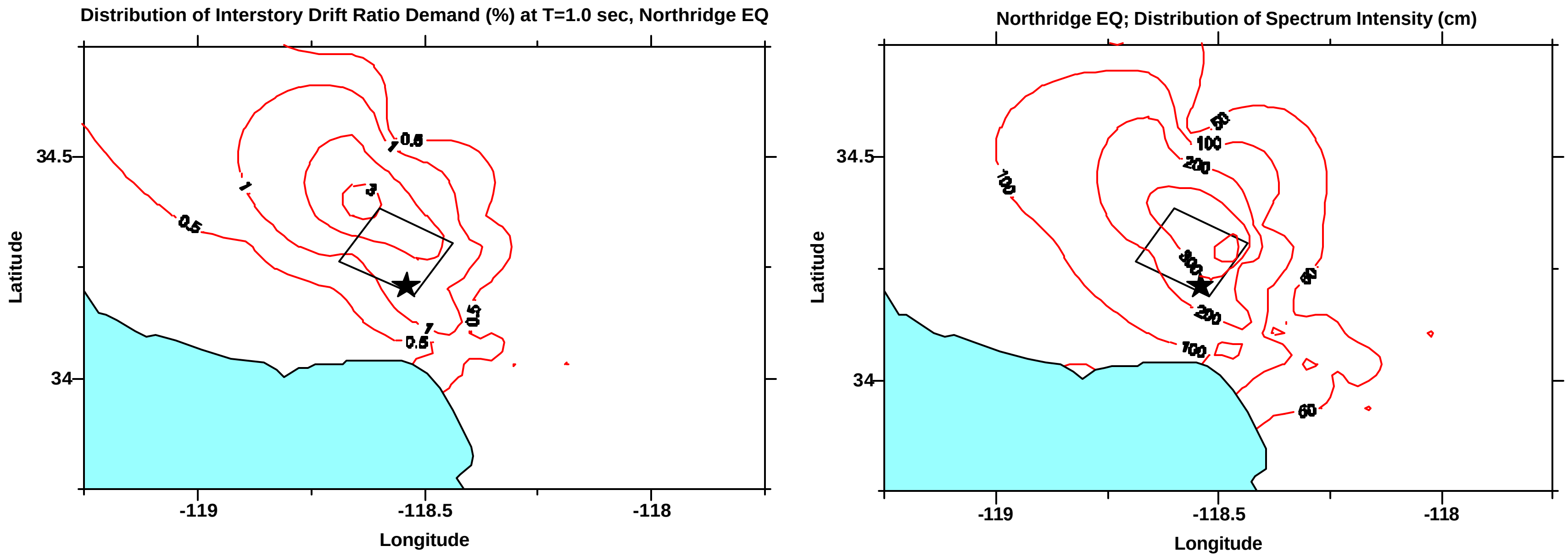


Figure 7: Distribution of interstory drift ratio demand (T= 1 sec); and spectrum intensity, Northridge earthquake.

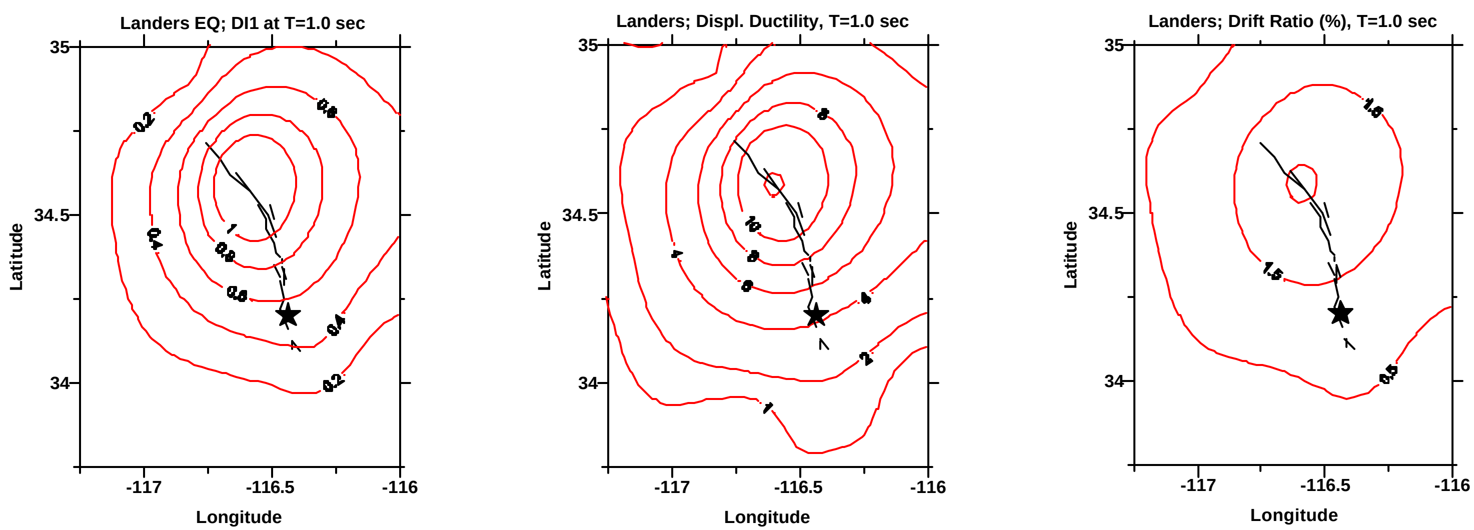


Figure 8: Distribution of damage spectral ordinates, displ. ductility and interstory drift ratio demands, Landers earthquake, T=1 sec.

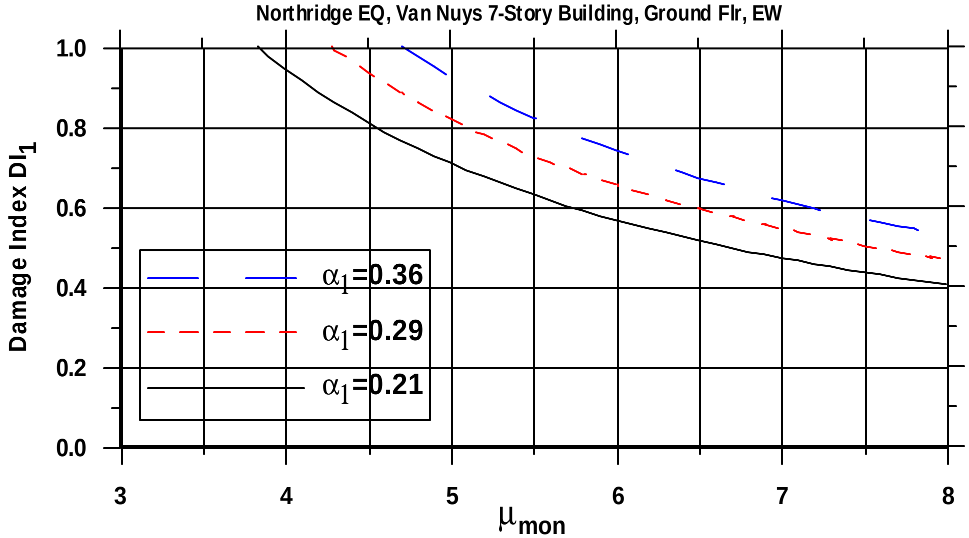


Figure 9: Van Nuys building, variation of damage index, based on ground floor (EW) acceleration record, Northridge earthquake.

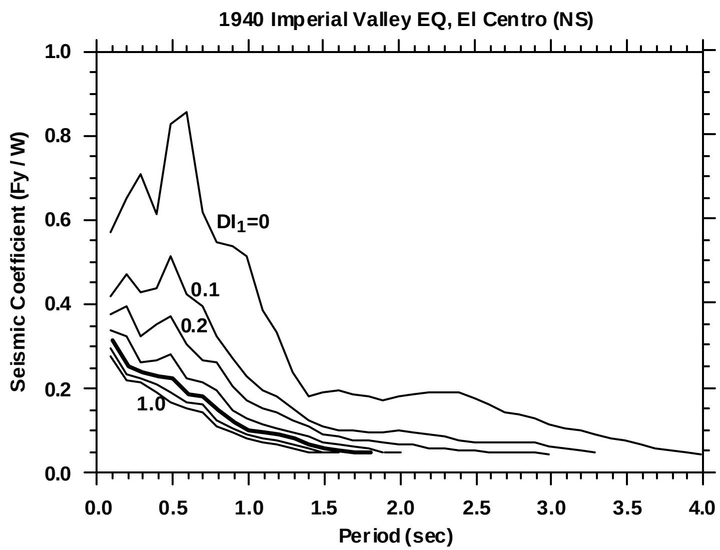


Figure 10: Strength spectra for constant values (0, 0.1, 0.2, 0.4, 0.6, 0.8, and 1.0) of damage index DI_1, for the 1940 El Centro (NS).

CORRELATION OF OBSERVED BUILDING PERFORMANCE WITH MEASURED GROUND MOTION

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Abstract

This paper describes the progress to date on the CSMIP-funded project to develop correlations of observed building performance with measured ground motion. Model development is in progress to develop motion-damage relationships in the form of damage probability matrices and fragility curves for wood frame, steel moment frame, rehabilitated unreinforced masonry, concrete frame, and concrete shear wall buildings – building types for which there are enough samples in the database to warrant statistical analysis. Two sample motion-damage relationships for wood frame dwellings that have been developed with the project data are included with example applications. As this project is in progress, all data and models discussed in this paper are preliminary and likely subject to revision at a later date.

Introduction

Relationships between building performance and ground motion form the core of earthquake loss estimation methodologies, and are also used for structural analysis studies and in the design code formulation process. Currently-used motion-damage relationships are based primarily on models developed from expert opinion, such as ATC-13 (ATC, 1985), or models that combine analytical model results with expert opinion, such as HAZUS99 (FEMA, 1999). Attempts have been made to update the published motion-damage relationships with empirical data collected after damaging earthquakes (see Anagnos et al., 1995). Small improvements have been made to models for specific building types, but typically with the use of proprietary insurance loss data with inferred ground motion information.

Following the 1994 Northridge earthquake, an effort was made to systematically document the effects of earthquake shaking on structures adjacent to locations of strong ground motion recordings. The ATC-38 project (ATC, 2000) involved the inspection of more than 500 buildings located near (within 1000 feet of) 30 strong motion recording stations. The resulting database of building characteristic and performance documentation, photos, and strong motion recordings provides a wealth of information for developing new motion-damage relationships based on non-proprietary empirical data. A similar dataset was also developed following the recent Chi-Chi, Taiwan earthquake.

The purpose of the CSMIP-funded project discussed in this paper is to develop motion-damage relationships based on the correlation of observed building performance with measured ground motion parameters. The project tasks include: identifying and collecting appropriate datasets; analyzing, interpreting, and archiving data records; developing motion-damage relationships in the form of damage probability matrices and fragility curves; and illustrating the use of the final relationships. The remainder of this paper discusses these tasks in more detail and the status of the project, which at the current time is approximately 65% complete.

Dataset Collection

In order to develop meaningful and useful motion-damage relationship that correlate building performance to recorded ground motion data, the datasets have to satisfy certain criteria, including:

- Proximity to free-field ground motion recordings – building should be located close enough to strong motion recordings so that the shaking at the building site can be approximated as the shaking at the instrument site. Also, the building should not have any site-specific geologic conditions that might alter the ground shaking at the site.
- Non-proprietary – the datasets should contain information that is available to the general public so that other researchers may use the raw data, with their own proprietary data or with information collected after future earthquakes.
- Sufficient number of data points –statistical relationships will only be meaningful for those building classes with a large enough sample size.
- Consistent building survey information – building performance data should have been collected in a standard format with consistent inspector interpretation of qualitative and quantitative measures of damage.
- Unbiased with respect to building damage – datasets often include information only for damaged buildings. Statistical relationships will not be meaningful unless the datasets include information for both damaged and undamaged buildings.

The first task of the project was to identify and collect datasets that meet the above criteria, which were found to be very stringent. The following datasets were collected for use in the project:

- ATC-38 – Database on the Performance of Structures Near Strong-Motion Recordings: 1994 Northridge, California Earthquake (ATC, 2000)
- LADiv88 – Rutherford and Chekene Database on the Performance of Rehabilitated Unreinforced Masonry Buildings (Retrofitted According to Los Angeles Division 88 Standards) in the 1994 Northridge, California Earthquake (Lizundia and Holmes, 1997)

- SAC –Database on the Performance of Steel Moment Frame Buildings in the 1994 Northridge, California Earthquake (FEMA, 2000)
- Chi-Chi – Degenkolb Database on the Performance of Buildings Near Strong-Motion Recording Stations (Heintz and Poland, 2001)

Following the collection of the building performance datasets, the accompanying strong ground motion data were identified and collected. For the ATC-38 and Chi-Chi building datasets, the strong ground motion data are included as database tables linked via the attribute containing the building identification number. All buildings in these two datasets could be used in the analysis as they are all located very close (within 1000 feet) of the recording stations.

For the SAC and LADiv88 building datasets, only those buildings located near to free-field strong motion recording stations (and on similar site conditions) were extracted from the complete databases. This was done by mapping the building locations in a GIS and overlaying a map of the ground motion recording stations. Two classes of buildings were extracted from their respective datasets – those within 1000 feet of a recording station and those within 1 km of a recording station. The 1000 foot criterion was the approximate distance used in the ATC-38 and Chi-Chi datasets. The 1 km criterion was added so that at a later date (depending on project schedule) a sensitivity study of the distance criterion can be done.

The strong ground motion data for the stations identified within the vicinity of the SAC and LADiv88 buildings were obtained from several sources including:

- COSMOS – Consortium of Organizations for Strong-Motion Observation Systems Virtual Data Center, which contains links to strong ground motion from the California Division of Mines and Geology, the U.S. Geological Survey, the U.S. Bureau of Reclamation, and the U.S. Army Corps of Engineers (www.cosmos-eq.org)
- PEER – Pacific Earthquake Engineering Research Center Strong Motion Database (peer.berkeley.edu/smcat/)
- NGDC – National Geophysical Data Center Earthquake Strong Motion CD-ROM (www.ngdc.noaa.gov)

Table 1 shows a general distribution of the buildings extracted from the datasets for use in the project.

Ground Motion Analysis

Several ground motion parameters were identified as potential candidates for correlation with building performance data. The parameters include those deemed relevant to the intended use of the resulting motion-damage relationships, i.e., they are typically computed in loss estimation and design procedures.

The following parameters have been computed and archived for each strong motion data record:

- Time history data parameters (maximum of two horizontal components, average of two horizontal components, and vertical):
  - Peak ground acceleration (*PGA*)
  - Peak ground velocity (*PGV*)
  - Peak ground displacement (*PGD*)
  - ShakeMap Instrumental Intensity (I_{mm})
    From Wald et al. (1999), computed as a function of *PGA* in cm/sec^2 and *PGV* in cm/sec according to the following:
    $$I_{mm} = 3.66 log(PGA) – 1.66 \qquad \text{for } I_{mm} < 7 \qquad (1a)$$
    $$I_{mm} = 3.47 log(PGV) + 2.35 \qquad \text{for } I_{mm} = 7 \qquad (1b)$$
  - Duration (T_d)
    For the total record
    For the time period bracketed by 90% of the cumulative energy
    For the time period bracketed by a 0.05g cut-off acceleration level
  - Root mean square acceleration (a_{RMS})
    Computed from the acceleration time history *a(t)* for the three time durations (T_d) listed above as follows:
    $$a_{RMS} = \sqrt{\frac{1}{T_d}\int_0^{T_d} a(t)^2\, dt} \qquad (2)$$
  - Arias Intensity (A_I)
    Computed from the acceleration time history *a(t)* for the three time durations (T_d) as follows:
    $$A_I = \int_0^{T_d} a(t)^2\, dt \qquad (3)$$
- Response spectra data parameters (maximum of two horizontal components, average of two horizontal components, and vertical):
  - Acceleration spectrum intensity (*ASI*)
    Computed as the area under the acceleration response spectrum between 0.1 and 0.5 seconds (Von Thum et al., 1988)
  - Effective peak acceleration (*EPA*)
    Computed as the average of the acceleration response spectrum between 0.1 and 0.5 seconds, divided by 2.5 (ATC, 1978)
  - Effective peak velocity (*EPV*)
    Computed as the average of the velocity response spectrum between 0.8 and 1.2 seconds, divided by 2.5 (ATC, 1978)
  - Housner intensity (S_I)
    Computed as the area under the pseudo velocity response spectrum between 0.1 and 2.5 seconds (Housner, 1952)

- Spectral acceleration at several periods ($S_a(T)$)
- Spectral velocity at several periods ($S_v(T)$)
- Spectral displacement at several periods ($S_d(T)$)

- Others (not computed, but acquired through map overlays in GIS software):
  - Modified Mercalli Intensity (*MMI*)
  - Site class at the recording station site using the 1997 NEHRP Classification (FEMA, 1997)

Table 1. Approximate Distribution of Building Data for Use in Model Development

Building Type	Number of Building Records	
	Within 1000 ft of station	**1000 ft - 1 km from station**
Wood Frame	270	
Steel Frame	102	57
Concrete Frame	104	
Concrete Shear Wall	73	
Reinforced Masonry	89	
Unreinforced Masonry (URM)	18	
Rehabilitated URM	54	116
Precast	10	
TOTAL	720	173

Building Response Analysis

The building response datasets were initially analyzed for two purposes – to group the buildings into similar structural classes and to interpret the damage survey information. The grouping of buildings by structural class was done according to the FEMA 310 (FEMA, 1998) model building types shown in Table 2. This classification is similar to that used in the ATC-38 database; however, an important difference is the inclusion of model building type W1A to account for multi-story, multi-unit residences with tuck-under parking. For several of the classes shown in Table 2, the number of data points (see Table 1) is not sufficient to develop motion-damage relationships for those classes. As discussed in the next section, relationships are being developed for wood frame, steel moment frame, rehabilitated unreinforced masonry, concrete frame, and concrete shear wall buildings.

The building performance information required standardization in terms of damage to structural and nonstructural components. The following classifications are used for structural and nonstructural (if available) damage or performance:

- ATC-13 (ATC, 1985) – Damage states are as follows:
  - 1 = None = 0% loss

- 2 = Slight = 0-1% loss
- 3 = Light = 1-10% loss
- 4 = Moderate = 10-30% loss
- 5 = Heavy = 30-60% loss
- 6 = Major = 60-100% loss
- 7 = Destroyed = 100% loss

- HAZUS99 (FEMA, 1999) – Damage states are as follows:
  - None = 0% loss
  - Slight = 2% loss
  - Moderate = 10% loss
  - Extensive = 50% loss
  - Complete = 100% loss

- Vision 2000 (SEAOC, 1995) – Performance levels are as follows:
  - Fully Operational = 9-10 = Negligible damage
  - Operational = 7-8 = Light damage
  - Life Safe = 5-6 = Moderate damage
  - Near Collapse = 3-4 = Severe damage
  - Collapse = 1-2 = Complete damage

- FEMA 273/274 (FEMA, 1997) – Performance levels are as follows:
  - Operational = Very light damage
  - Immediate Occupancy = Light damage
  - Life Safety = Moderate damage
  - Collapse Prevention = Severe damage

In addition to the standardization of the structural classes and performance descriptions, the design code year and fundamental period were added to the database attributes associated with each building. The design code year is used to compute the design base shear (in terms of the seismic coefficient) and roof drift limit for each building. The fundamental period is used to compute the demand spectral values as described later in this section. For the general building types, the fundamental period is computed as a function of building height, *H*, as follows:

Wood frame building, based on Camelo et al. (2001):

$$T = 0.032H^{0.55} \tag{4}$$

Steel frame buildings, based on Chopra et al. (1998):

$$T = 0.035H^{0.80} \tag{5}$$

Reinforced concrete frame buildings, based on Chopra et al. (1998):

$$T = 0.018H^{0.90} \tag{6}$$

Rehabilitated unreinforced masonry buildings, based on UBC 1997 (ICBO, 1997):

$$T = 0.020H^{0.75} \tag{7}$$

Concrete shear wall buildings, based on UBC 1997 (ICBO, 1997):

$$T = 0.020H^{0.75} \tag{8}$$

Table 2. Model Building Types (from FEMA, 1998)

W1: Wood Light Frames	
W1	Single or multiple family dwellings
W1A	Multi-story, multi-unit residences with open front garages at the first story
W2: Wood Frames, Commercial and Industrial	
S1: Steel Moment Frames	
S1	Stiff diaphragms
S1A	Flexible diaphragms
S2: Steel Braced Frames	
S2	Stiff diaphragms
S2A	Flexible diaphragms
S3: Steel Light Frames	
S4: Steel Frame with Concrete Shear Walls	
S5: Steel Frame with Infill Masonry Shear Walls	
S5	Stiff diaphragms
S5A	Flexible diaphragms
C1: Concrete Moment Frames	
C2: Concrete Shear Wall Buildings	
C2	Stiff diaphragms
C2A	Flexible diaphragms
C3: Concrete Frame with Infill Masonry Shear Walls	
C3	Stiff diaphragms
C3A	Flexible diaphragms
PC1: Precast/Tiltup Concrete Shear Walls	
PC1	Stiff diaphragms
PC1A	Flexible diaphragms
PC2: Precast Concrete Frame	
PC2	Stiff diaphragms
PC2A	Flexible diaphragms
RM1: Reinforced Masonry Bearing Wall with Flexible Diaphragms	
RM2: Reinforced Masonry Bearing Wall with Stiff Diaphragms	
URM: Unreinforced Masonry Bearing Wall	
URM	Stiff diaphragms
URMA	Flexible diaphragms

The seismic demands on the building, in terms of displacement and base shear, have also being computed for each building in the dataset. This will allow for development and evaluation of relationships relating earthquake performance, not only to the recorded and computed ground motion parameters listed in the previous section, but also to seismic demand levels.

The estimate of building displacement demand during the recorded earthquake ground motion is computed as the spectral displacement demand normalized by the height of the building to obtain a *spectral drift ratio*. The spectral drift ratio, d_{S_d}, is calculated by the following:

$$d_{S_d} = S_d(T)/H \tag{9}$$

where $S_d(T)$ is the building spectral displacement demand obtained from the 5% damped response spectrum of the earthquake ground motion recorded at or near the building site, and H is the building height.

A minor inconsistency occurs when calculating the spectral drift ratio by Equation 9, due to the fact that the spectral displacement demand, based on an equivalent single degree-of-freedom system (SDOF), is normalized by the building height instead of an equivalent height of the SDOF system. In order to achieve consistency and also so that the demands can be compared to building code drift limits and FEMA 273 drift ratios related to building performance, the spectral drift ratio calculated in Equation 9 can be translated to an estimate of the building *roof drift ratio*. The roof drift ratio, d_R, is calculated by the following:

$$d_R = d_{S_d} C_0 = \frac{S_d(T) C_0}{H} \tag{10}$$

where C_0 is a modification factor that translates the spectral displacement demand, which represents the displacement of an equivalent SDOF system, to the roof displacement of the building. The value of C_0 depends on the dynamic characteristics of the building, and is based on the values provided by FEMA 273.

An effective measure of the ratio of building base shear demand to the building weight is the spectral acceleration, $S_a(T)$. The spectral acceleration is obtained from the 5% damped response spectrum of the earthquake ground motion recorded at or near the building site.

Preliminary Model Development

Motion-damage relationships are currently being developed in two forms – damage probability matrices (DPM) and fragility curves. DPMs show the conditional probability of being in a discrete damage state as a function of the input ground motion level, which can be a discrete value (e.g., *MMI*) or a range of values (e.g., *PGA*). The DPMs developed from the datasets are being fit to conditional probability distributions, typically Beta or lognormal distributions, although others will also be tested. Figure 1 illustrates with hypothetical data how the probability distribution corresponds to one column of the DPM. Fragility curves show the conditional probability of being equal to or exceeding a given damage state as a function of the ground motion parameter. Fragility curves are related to the DPMs and can be computed from them as shown, as they are essentially curves of the cumulative probability distribution for each damage state as a function of the ground motion level. Figure 1 shows how a fragility curve corresponds to a row (and the sum of the rows below) in the DPM.

DAMAGE PROBABILITY MATRIX					
	Probability (%) of being in a given damage state as a function of PGA for a given building type				
Damage State	PGA 0-0.2g	PGA 0.2-0.4g	PGA 0.4-0.6g	PGA 0.6-0.8g	PGA 0.8-1.0g
1, <2% loss	90	80	60	25	20
2, 2-10% loss	10	15	20	40	30
3, 10-30% loss	0	5	15	20	30
4, 30-60% loss	0	0	5	10	15
5, >60% loss	0	0	0	5	5

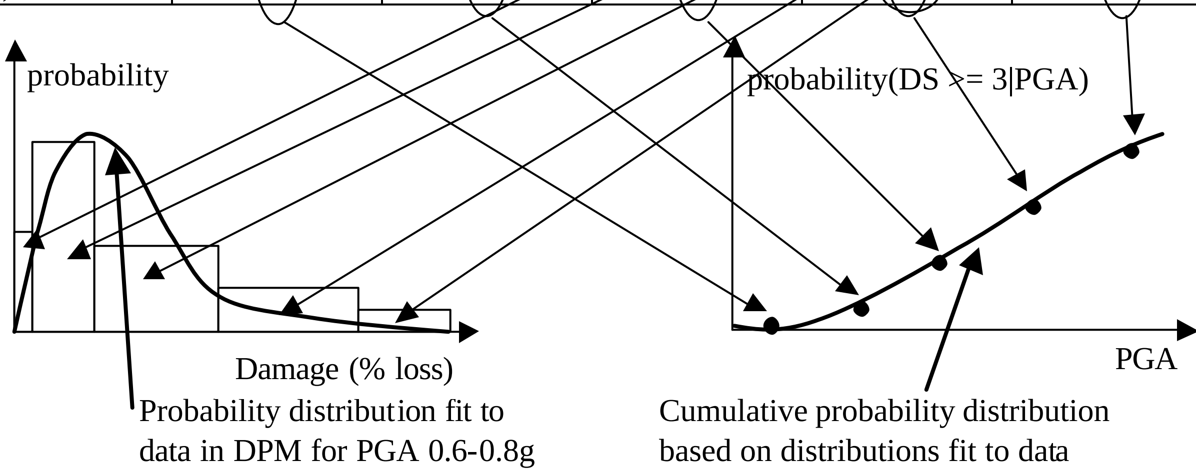

Figure 1. Illustration of DPM, probability distribution fit, and fragility curve.

In developing the motion-damage relationships, it is important to consider the relative uncertainty in the quality of the data, as the datasets have varying degrees of reliability and in some cases missing attributes had to be inferred from the reported data. This issue is being addressed when combining the data from the various sources by assigning weighting or quality factors to the data. Statistical analysis software is being utilized to aid in the development of the motion-damage relationships.

The final motion-damage relationships will be compared to those that are published in the literature (e.g., ATC-13 and HAZUS99). The comparison will include a discussion of how some of the motion-damage relationships developed in this project can be used to update current loss estimation models.

Example Models and Application

This section includes two motion-damage relationships that were developed from the data for wood frame dwellings (type W1). These relationships should be considered preliminary as the project is still in progress and the model development task has not yet been completed. The models presented here illustrate one form (DPM) of the relationships being developed, and their use for regional loss estimation and site-specific building evaluation.

The first model shows the relationship between building damage in terms of ATC-13 structural damage state and ShakeMap instrumental intensity, I_{mm}. The damage probability matrix shown in Table 3 gives, for each range in I_{mm}, the probability of being in one of seven damage states. The ATC-13 damage states each have an associated percent loss (in terms of percentage of replacement cost) as shown earlier in this paper. Using the mean of the range in percent loss for each damage state (termed the "central damage factor" in ATC-13) and the probabilities of being in each damage state, the expected loss can be computed for each range in I_{mm} as shown in Table 3.

Table 3. Preliminary Damage Probability Matrix for Wood Frame Dwellings (W1)

Damage State	Range in ShakeMap Intensity (I_{mm})					
	6-6.5	6.5-7.0	7-7.5	7.5-8	8-9	9-10
1	0.34	0.59	0.44	0.00	0.15	0.27
2	0.64	0.39	0.47	1.00	0.70	0.55
3	0.00	0.00	0.06	0.00	0.15	0.03
4	0.02	0.00	0.03	0.00	0.00	0.09
5	0.00	0.00	0.00	0.00	0.00	0.00
6	0.00	0.02	0.00	0.00	0.00	0.06
7	0.00	0.00	0.00	0.00	0.00	0.00
Expected Loss (%)	0.72	2.01	1.12	0.50	1.11	7.09

Figure 2 shows the application of the damage probability matrix given in Table 3. A ShakeMap showing Intensity (I_{mm}) computed for the 1994 Northridge earthquake in shown in Figure 2a. Figure 2b shows the distribution of expected loss to wood frame dwellings (W1) based on the data given in Table 3. This example illustrates the utility of a motion-damage relationship for making first order rapid estimate of earthquake damage immediately after an event.

The second model shows the relationship between building performance in terms of the FEMA 273 performance levels listed earlier in this paper and the roof drift ratio. The damage probability matrix shown in Table 4 gives, for each range in roof drift ratio, the probability of being in one of four performance states.

An example application of the damage probability matrix shown in Table 4 is for a site-specific performance-based building evaluation. For instance, using the FEMA 273 methodology, the site response acceleration spectra for the 10% in 50 year and 2% in 50 year hazard levels are as shown in Figure 3. Given a 2-story (H = 24 feet) wood frame dwelling with a fundamental period of 0.184 sec (computed according to Equation 4), the spectral acceleration values are obtained from Figure 3 and converted to spectral displacement values. Roof drift ratios are computed according to Equation 10, using $C_0 = 1.2$. Using the relationship shown in Table 4, the probability of the various performance levels is computed for the two hazard levels, thus giving a first order approximation of the likelihood of meeting the performance objectives for the building (e.g., immediate occupancy for both the 2% in 50 year and 10% in 50 year ground shaking hazard levels). Table 5 shows the results of this exercise.

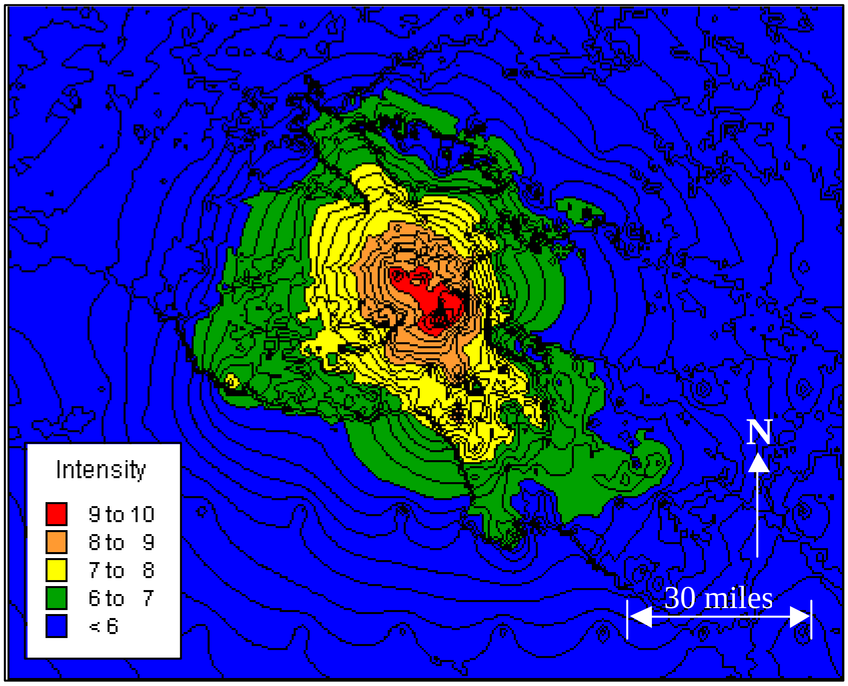

Figure 2a. ShakeMap showing Intensity distribution for the 1994 Northridge earthquake.

The model development phase of the project is in progress. The two models included in this section represent a very small sample of the motion-damage relationships that are being developed as part of this project. As these models are preliminary and subject to revision, comments will not be made in this paper with respect to the appropriateness of the models for the applications illustrated here, the uncertainty, the comparison with other published models, and other discussion points that will be included in the final project report.

Summary

This paper describes the progress to date on the CSMIP-funded project to develop correlations of observed building performance with measured ground motion. The necessary datasets have been collected, screened, and archived. Ground motion and building response parameters have been computed. Model development is in progress to develop motion-damage relationships in the form of damage probability matrices and fragility curves for wood frame, steel moment frame, rehabilitated unreinforced masonry, concrete frame, and concrete shear wall buildings – building types for which there are enough samples in the database to warrant statistical analysis.

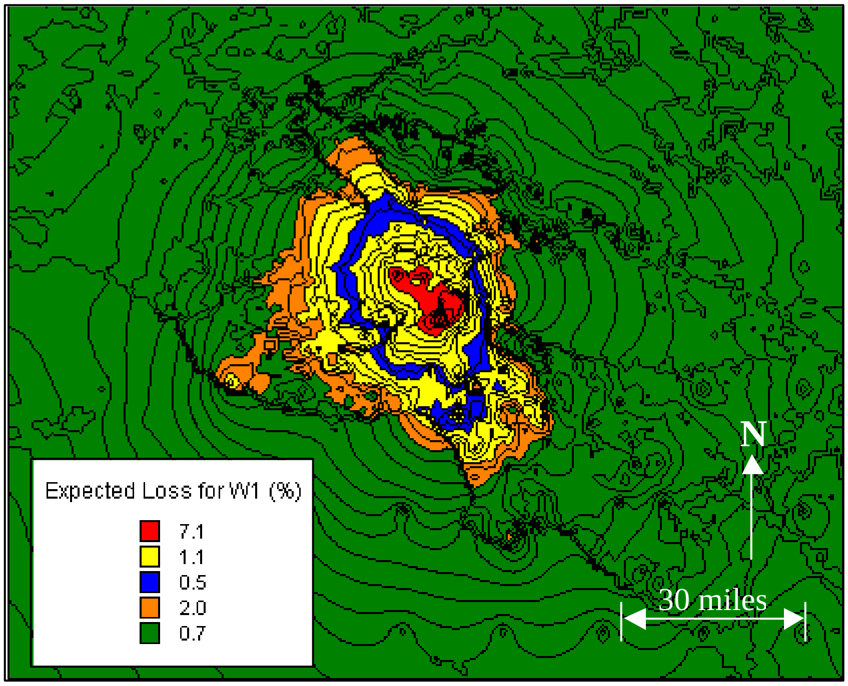

Figure 2b. **ShakeMap showing distribution of expected loss for wood frame dwellings in the 1994 Northridge earthquake based on data in Table 3.**

Table 4. **Preliminary Damage Probability Matrix for Wood Frame Dwellings (W1)**

Performance Level	Range in *Roof Drift Ratio* (%)				
	< 0.045	0.045-0.06	0.06-0.075	0.075-0.12	> 0.12
Operational	0.34	0.59	0.44	0.00	0.15
Immediate Occupancy	0.64	0.39	0.47	1.00	0.70
Life Safety	0.00	0.00	0.06	0.00	0.15
Collapse Prevention	0.02	0.00	0.03	0.00	0.00

Two sample motion-damage relationships for wood frame dwellings that have been developed with the project data are included. The use of these relationships is illustrated by application to regional rapid loss estimation and site-specific building evaluation. As this project is in progress and not scheduled to be completed for another six months, all data and models discussed in this paper are preliminary and likely subject to revision at a later date.

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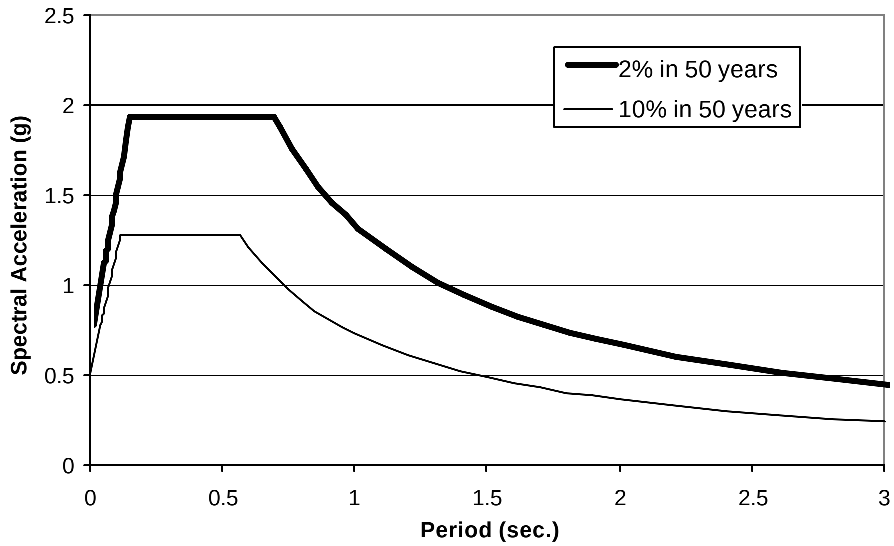

Figure 3. Example site-specific acceleration response spectra.

Table 5. Results of Example Site-Specific Building Evaluation

	2% in 50 year Hazard	**10% in 50 year Hazard**
Spectral Acceleration (g)	1.92	1.28
Spectral Displacement (in.)	0.623	0.416
Roof Drift Ratio (%)	0.26	0.17
P(Operational Performance)	0.15	0.15
P(Immediate Occupancy)	0.70	0.70
P(Life Safety)	0.15	0.15
P(Collapse Prevention)	0.00	0.00

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ANALYSIS OF PACOIMA DAM USING RECENTLY RECORDED SEISMIC MOTIONS: REPORT ON PROGRESS

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Abstract

Ground and structure motions were recorded on 17 channels at Pacoima Dam during a magnitude 4.3 earthquake on January 13, 2001. These data are examined by system identification and finite element modeling to quantify the effects of nonuniform ground motion and the level of damping exhibited by the dam. The system identification study indicates the dynamic response of the dam is due mostly to two modes which have frequencies close to 5hz and damping in the range of 4% to 7% of critical. The displacement histories of the dam are reproduced fairly well by the finite element model, but differences in amplitudes for some of the channels remain to be resolved. In addition, more work is needed to obtain confidence in the results, especially the relatively high level of damping which seems to be present.

Introduction

Pacoima Dam is a 113m high concrete arch dam located north of the city of Los Angeles. A well known ground motion record obtained above the south abutment during the 1971 San Fernando earthquake showed large accelerations of 1.25g horizontal and 0.70g vertical which have been attributed to topographical amplification. A more extensive 17-channel accelerograph array was installed later and was in place during the 1994 Northridge earthquake. Nine of these channels were located on the dam-rock interface in order to capture the spatially nonuniform features of the seismic input, and the remaining eight channels were located either on the dam crest or at 80% height on the downstream face of the dam. Unfortunately, middle portions of many of the 1994 records contained off-scale high frequency motions which could not be digitized. These motions probably resulted from impacts in the contraction joints between the blocks of the dam and at a thrust block on the south abutment. Movement of a rock mass occurred on the south abutment in both the 1971 and 1994 earthquakes, more severely in the latter. These movements opened and reopened a gap in the joint at the south thrust block. Repairs undertaken after both earthquakes included stabilization of the damaged abutment and filling the gap at the thrust block.

This paper is a progress report on an investigation into a set of records obtained at Pacoima Dam during a magnitude 4.3 earthquake on January 13, 2001 (CSMIP, 2002). By the time of this earthquake, the 17 analog channels at Pacoima Dam had been replaced by digital ones, and so the recent data is of high quality. Of principal interest in this study are the effects of non-uniformity in the ground motion and the level of damping present in the dam vibration. Both of these are important considerations in the earthquake response of dams, and neither has been adequately quantified to date.

The three sections that follow present a description of the records, an identification study in which best-fit parameters of a 2-mode linear system are determined, and an effort to calibrate a detailed finite element model using the recorded data. Future studies will employ the calibrated finite element model and stronger ground motions to evaluate the effects of cracking and joint opening on the dam response. The final report will also include an examination of records obtained on Pacoima Dam during the 1994 Northridge earthquake, which have been previously studied by others (Mojtahedi and Fenves, 2000).

Description of Records

Locations of the 17 channels at Pacoima Dam are shown in Figure 1. Channels 1 to 8 are on the body of the dam: six of these are radial in the horizontal plane with one each oriented tangentially and vertically. Channels 9 to 17 are located on the dam-rock interface in order to define the input motion to the dam. There are three channels each at stations on the north and south abutments and at the base. Acceleration and displacement time histories from the 2001 earthquake for all channels except vertically oriented ones are plotted in Figures 2 and 3. Peak values of acceleration, velocity and displacement are listed in Table 1 for each channel. The highest accelerations at the interface and on the dam are 0.10g and 0.16g, respectively. Since the level of shaking is much lower than during the 1994 Northridge earthquake, the acceleration records show none of the off-scale high frequency motions which characterized the Northridge earthquake accelerograms.

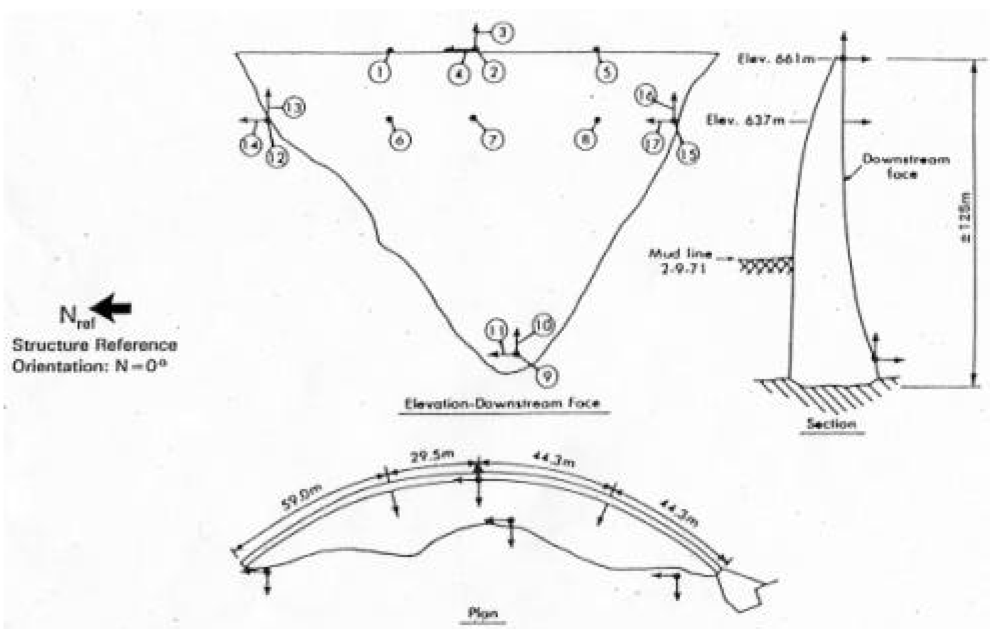

Figure 1. Location of the 17 recording channels at Pacoima Dam (CSMIP, 2002).

Although the intensity of the input motion from the 2001 earthquake is not high, the amplitude and phase variations around the canyon are probably more functions of frequency than amplitude, which means these characteristics of nonuniform input should be fully contained in the data and so can still be quantified. The moderate motions of the dam imply that it is vibrating in or near its linear range, and this simplifies the present study of the effects of nonuniform ground motion. Regarding damping, the data should lead to values appropriate for low-level vibration, which will be useful as a baseline. Another simplification arises from the water level being 41m below the crest during the 2001 earthquake, which is low enough so that it has only a minor effect on the dam response.

The recorded motions from the 2001 earthquake on the north and south abutments are of higher amplitude than those at the base. This amplification is shown in Figure 4 as a function of vibration frequency where plots of ratios of response spectra computed from the respective components of the abutment and base motions are presented. The most amplification is seen in the north-south component (close to cross-stream direction) at the south abutment, which is where the damage occurred in previous earthquakes. However, the other two components on the south abutment are amplified about the same, or even less, than the respective ones on the north abutment. At the fundamental frequency of the dam, which is shown to be about 5hz in the next section (actually two frequencies near 5hz), the amplification on the south abutment in the north-south direction is about 3.5, and for the other channels, the amplification ranges from 2 to 3. Amplification factors above four occur for two channels: the north-south component on the south abutment at frequencies around 4hz and 10hz, and the east-west component (close to the upstream-downstream direction) on the north abutment around 7hz.

Another aspect of the non-uniformity in the input motions is time shift. This quantity can be found between any two motions by integrating their product as a function of time offset between the two. The offset for which this correlation integral is maximum is the time shift, which is listed in Table 2 for respective components of the motions from the base station to the two abutment stations. As seen, the abutment motions in the horizontal directions lag (positive time shift) the base motions by times ranging from 41 to 66 milliseconds. Time shifts for the vertical component are smaller. Although the largest time shifts are a significant fraction of the fundamental period of the dam, which is about 200 milliseconds, the shifts are not expected to affect the dam response in a major way. This is because the horizontal abutment motions excite most of the dam response, and they are actually time shifted relatively little with respect to each other. Some further work is in progress to quantify the frequency dependence of the time shifts.

A long range goal of collecting ground motion data at the base and sides of canyons, as at Pacoima Dam, is to develop rules for prescribing nonuniform seismic input in safety assessment analyses of dams. Based on the data presented here, one could propose that time shift be a function of elevation and shear wave speed in the rock. For Pacoima Dam, using the 84m elevation difference between the base and abutment recording stations and a shear wave velocity for rock of 1500 to 2000 m/sec (see section on Finite Element Model Calibration) results in a time shift of 0.042 to 0.056 milliseconds, which is in the range of that found for the horizontal components of

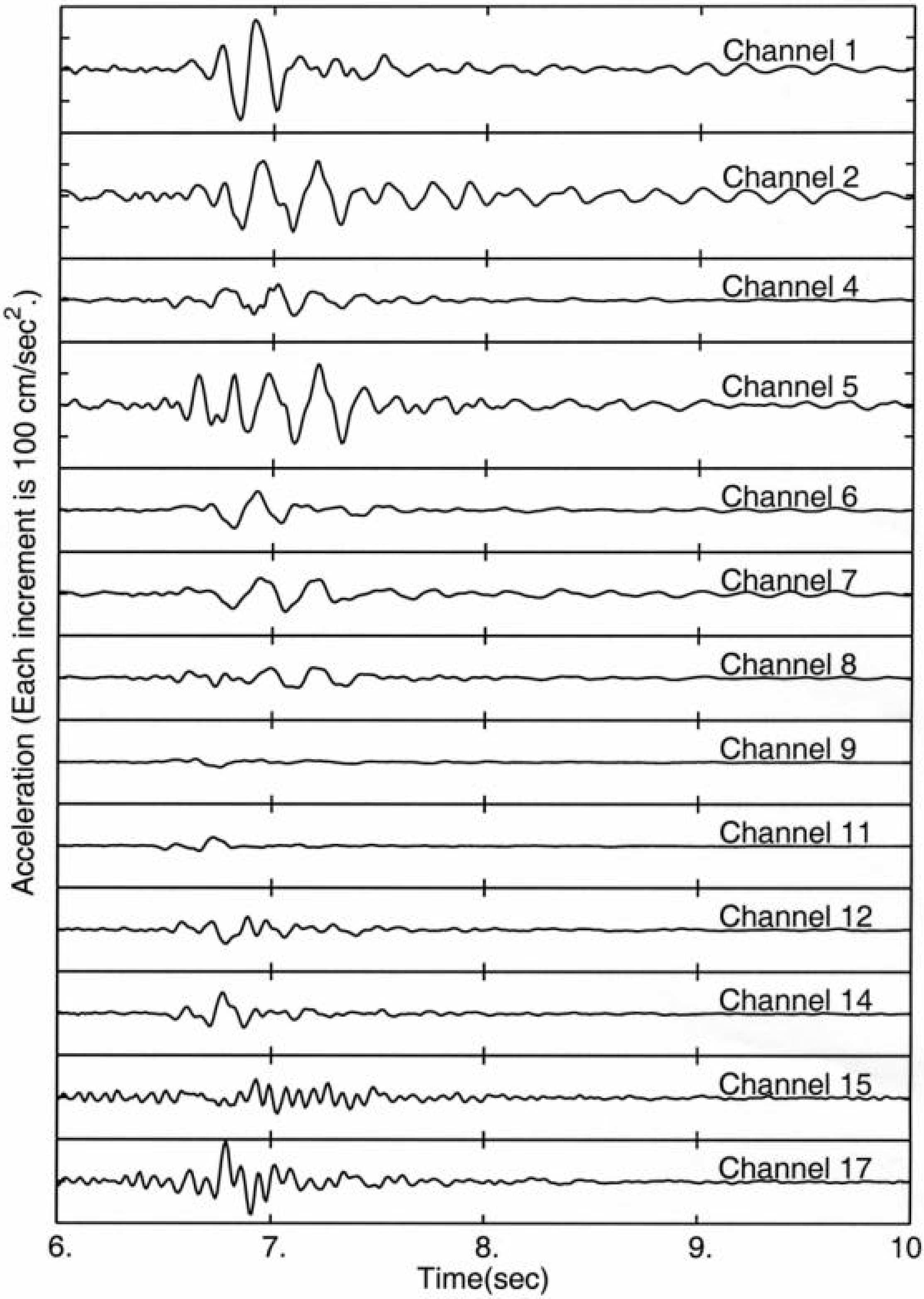

Figure 2. Horizontal components of acceleration recorded at Pacoima Dam on January 13, 2001.

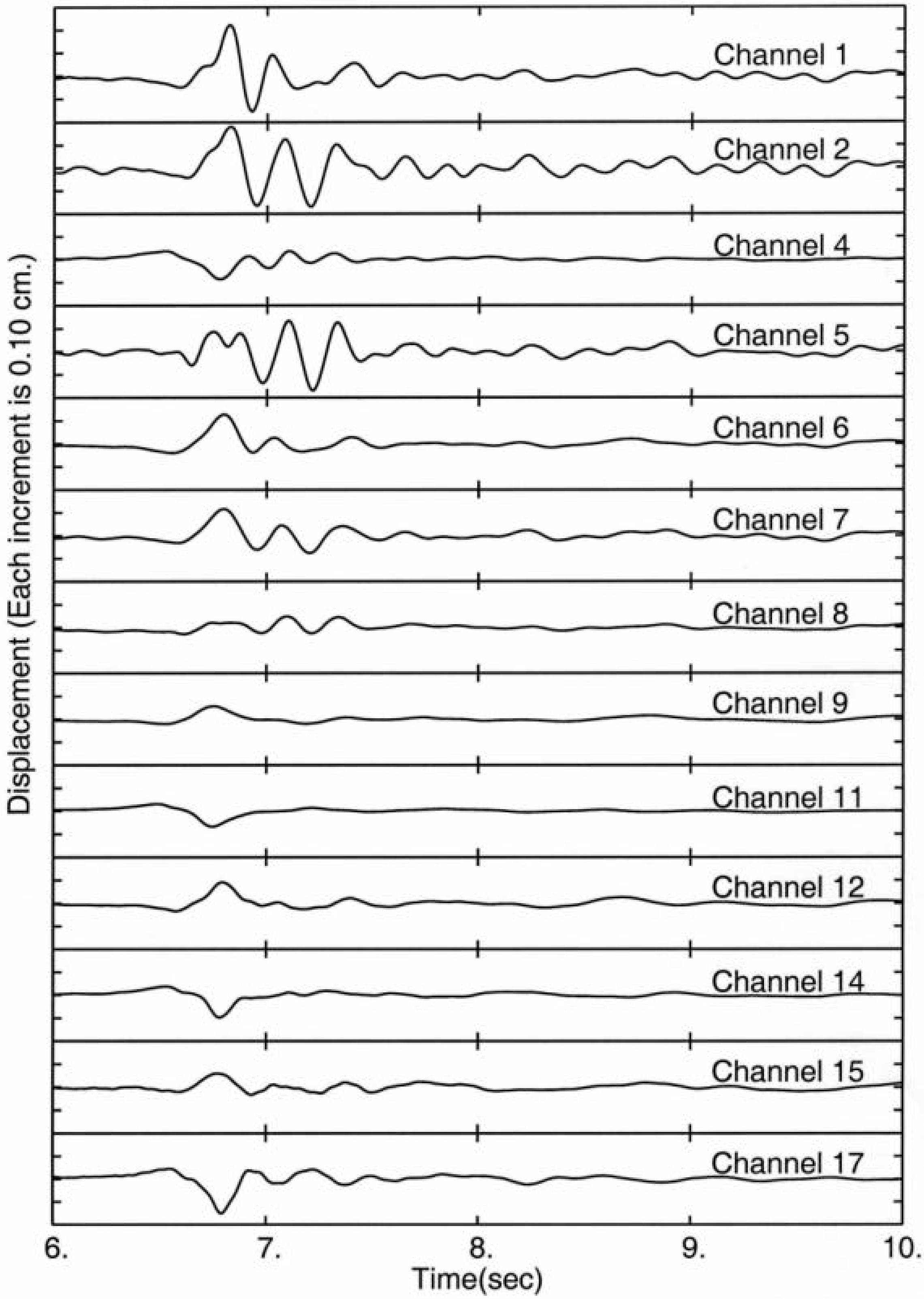

Figure 3. Horizontal components of displacement recorded at Pacoima Dam on January 13, 2001.

Channel	Location/Orientation	Acceleration	Velocity	Displacement
1	crest at north thrd point/radial	-0.16g	-6.2 cm/sec	0.22 cm
2	center crest/radial	-0.12g	-4.7 cm/sec	0.18 cm
3	center crest/up	-0.02g	0.6 cm/sec	0.02 cm
4	center crest/tangential	0.04g	1.3 cm/sec	-0.09 cm
5	crest at south qtr point/radial	0.13g	-3.9 cm/sec	-0.17 cm
6	80% height at north thrd pt/radial	0.05g	-2.0 cm/sec	0.13 cm
7	80% height at center/radial	-0.04g	-1.7 cm/sec	0.12 cm
8	80% height at south qtr pt/radial	0.02g	-1.1 cm/sec	0.05 cm
9	base/west	-0.01g	0.6 cm/sec	0.06 cm
10	base/up	0.01g	-0.2 cm/sec	0.02 cm
11	base/north	0.02g	-0.9 cm/sec	0.07 cm
12	north abutment/west	-0.03g	1.2 cm/sec	0.09 cm
13	north abutment/up	-0.01g	-0.5 cm/sec	0.03 cm
14	north abutment/north	0.05g	1.5 cm/sec	-0.10 cm
15	south abutment/west	0.04g	-0.9 cm/sec	0.06 cm
16	south abutment/up	0.02g	-0.3 cm/sec	0.03 cm
17	south abutment/north	0.10g	2.3 cm/sec	-0.15 cm

Table 1. Peak values of acceleration, velocity and displacement from the 17 channels on Pacoima Dam during the 2001 earthquake.

	East-west comp	Vertical comp	North-south comp
Base to north abutment	0.049 sec	0.024 sec	0.048 sec
Base to south abutment	0.041 sec	-0.008 sec	0.066 sec

Table 2. Time shifts from the base station to the stations on north and south abutments for east-west, vertical and north-south components of the 2001 earthquake records. Lag is positive.

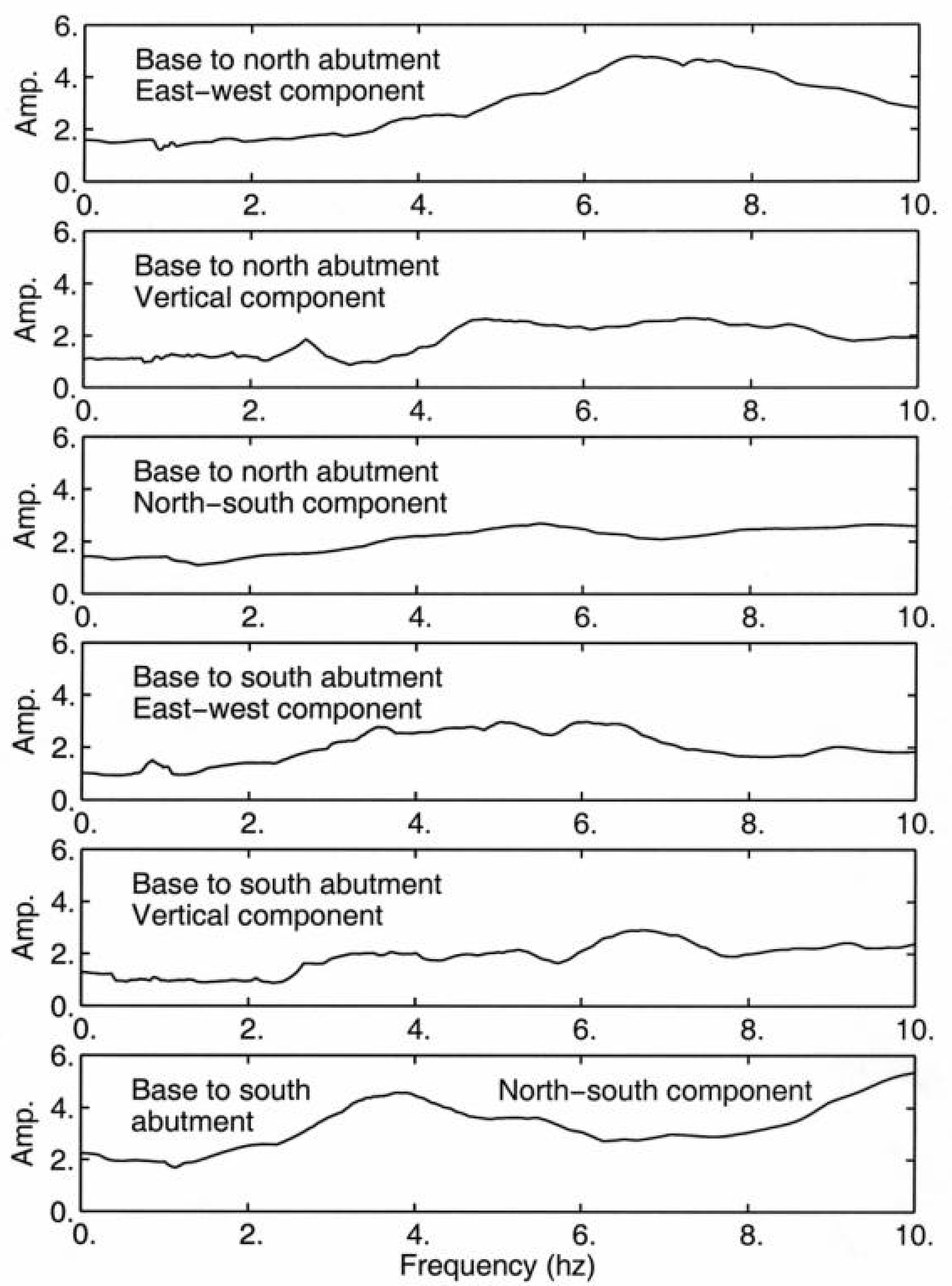

Figure 4. Amplification on the abutments of Pacoima Dam referred to motion at the base of the dam in terms of ratios of response spectra (5% damping).

ground motion (Table 2). An additional rule expressing amplification as a function of frequency and elevation could be formulated by averaging the results shown in Figure 4. Such rules would be applied to components of a reference motion to generate a suite of motions around a canyon. A major difficulty is how to select the reference motion. Should it be considered representative of a bottom site, in which case it would be amplified and time lagged up the canyon, or a site near the crest of the dam, in which case it would be de-amplified and time forwarded down the canyon, or somewhere in between? If the reference motion is to be selected by current standard procedures, this becomes an interesting question.

System Identification

System identification is performed using the computer program MODEID (Beck and Jennings, 1980) which models the structure as a linear system in modal coordinates excited by spatially nonuniform ground motion. To run MODEID, the user specifies the number of modes to be included and supplies the motions recorded on the structure (channels 1 to 8 in the present case) and at the "base" of the structure (channels 9 to 17 in the present case). Using the “base” motions as input, the program determines the frequencies, damping, shapes and participation factors for each mode, as well as the pseudo-static response matrix, which produce the best fit to the recorded structural motions.

For a two-mode solution, MODEID converged to modal frequencies of 4.8hz and 5.1hz with damping of 6.8% and 4.6% of critical, respectively. As shown from experience in forced vibration testing (Hall, 1988), arch dams typically have two closely spaced modal frequencies, and these correspond to mode shapes which can be classified as symmetric and antisymmetric. Previous forced vibration tests on Pacoima Dam in 1980 revealed frequencies of 5.4hz and 5.6hz (ANCO Engineers, 1982), higher than the MODEID values. Reasons for this difference are not clear. Damping values from the 1980 tests were also higher than those from MODEID, but data from those tests were of poor quality and this made it difficult to accurately determine damping. However, even the 4% to 7% damping from MODEID seems on the high side compared to forced vibration results from other dams (for example, 1.4% to 4.0% at Morrow Point Dam and 1.8% to 3.1% at Monticello Dam; see Hall, 1988). And it should be noted that MODEID identifies the fixed-base modal properties, ie, foundation interaction is not represented, and so there is no energy radiation from this source.

Figure 5 plots the recorded accelerations from channels 1, 2, 4 and 5 along with the best-fit responses of the MODEID model. The agreement is very good. However, because MODEID does not enforce "realistic" conditions in its parameter determination, the identified mode shapes appeared to violate orthogonality, and they did not match the expected symmetric and antisymmetric profiles. In addition, some terms of the pseudo-static response matrix were much too large. Because of these problems, a question arises about the accuracy of the damping values. (Modal frequencies are well determined.) In fact, there is a kind of ill-posedness between modal damping and participation factor, as a higher value of one can be significantly compensated by a lower value of the other.

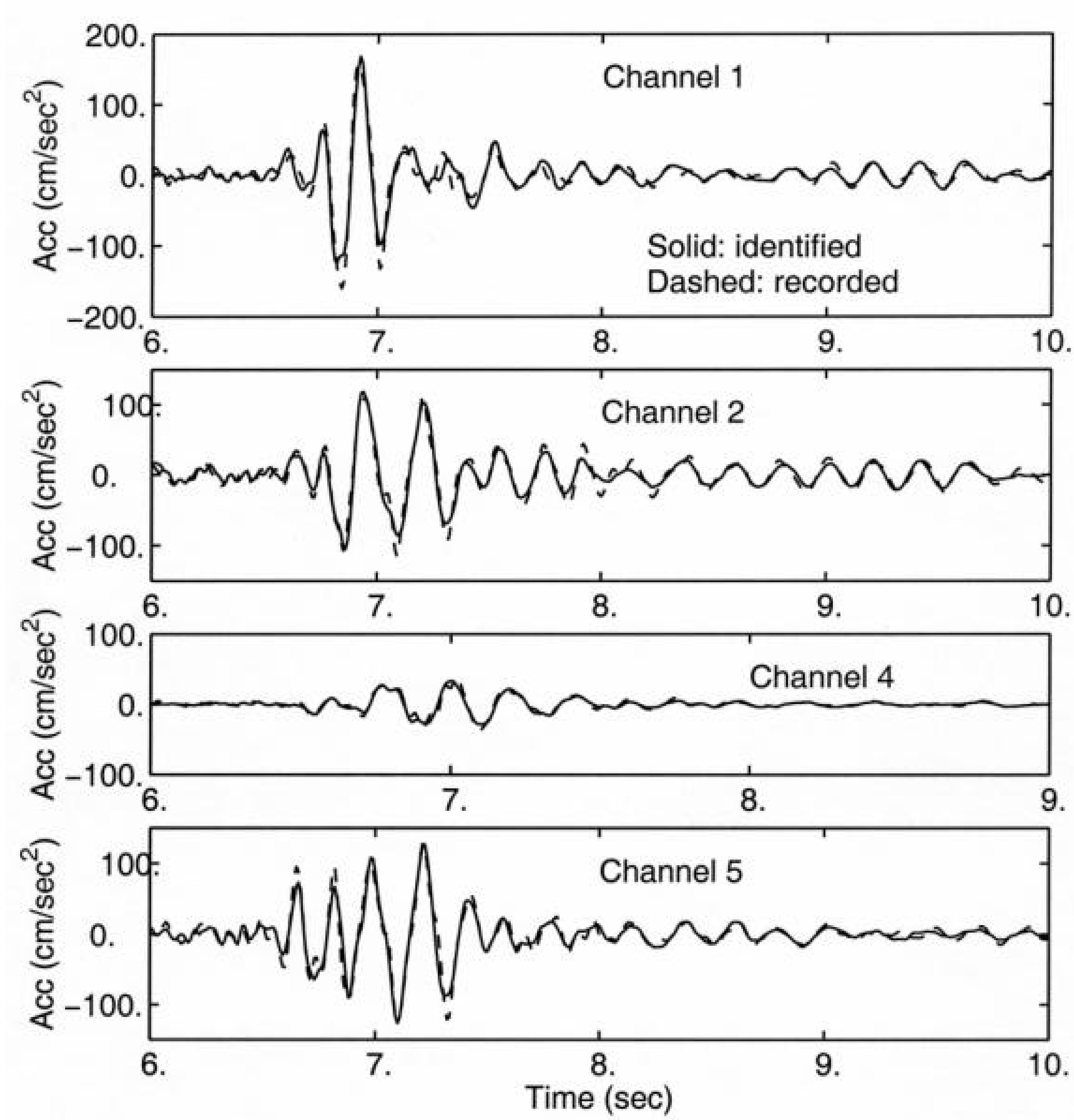

Figure 5. Comparison between the recorded accelerations at channels 1, 2, 4, and 5 and the best-fit accelerations from the MODEID system identification study.

More work on system identification is planned, especially to achieve confidence in the damping. First, sensitivity of the quality of the fit to the damping values will be examined. Second, the terms of the pseudo-static response matrix will be computed by the finite element program discussed in the next section and input as set quantities to MODEID. Constraining MODEID in this way should result in more realistic values for the remaining parameters to be determined. It should also be noted that the input motion is known to MODEID only at three locations, which could be a source of error. So, a third task is to investigate this issue through simulations using the finite element model.

Finite Element Model Calibration

A finite element model of Pacoima Dam, massless rock foundation, and incompressible water reservoir was constructed with the computer program SCADA (Hall, 1996). Shell elements are used for the dam, solid brick elements for the foundation, and pressure brick elements for the water (Figure 6). Rayleigh damping is employed using the stiffness and mass matrices of the dam and the stiffness matrix of the rock to construct a proportional damping matrix. The foundation model is connected only to the dam, and for modeling purposes the south thrust block is considered to be part of the foundation. Nodes of the water mesh are fixed down to the surface elevation at the time of the 2001 earthquake. SCADA uses the smeared crack method to model opening, closing and sliding nonlinearity associated with contraction joints and cracks in the dam, or it can operate in a linear mode.

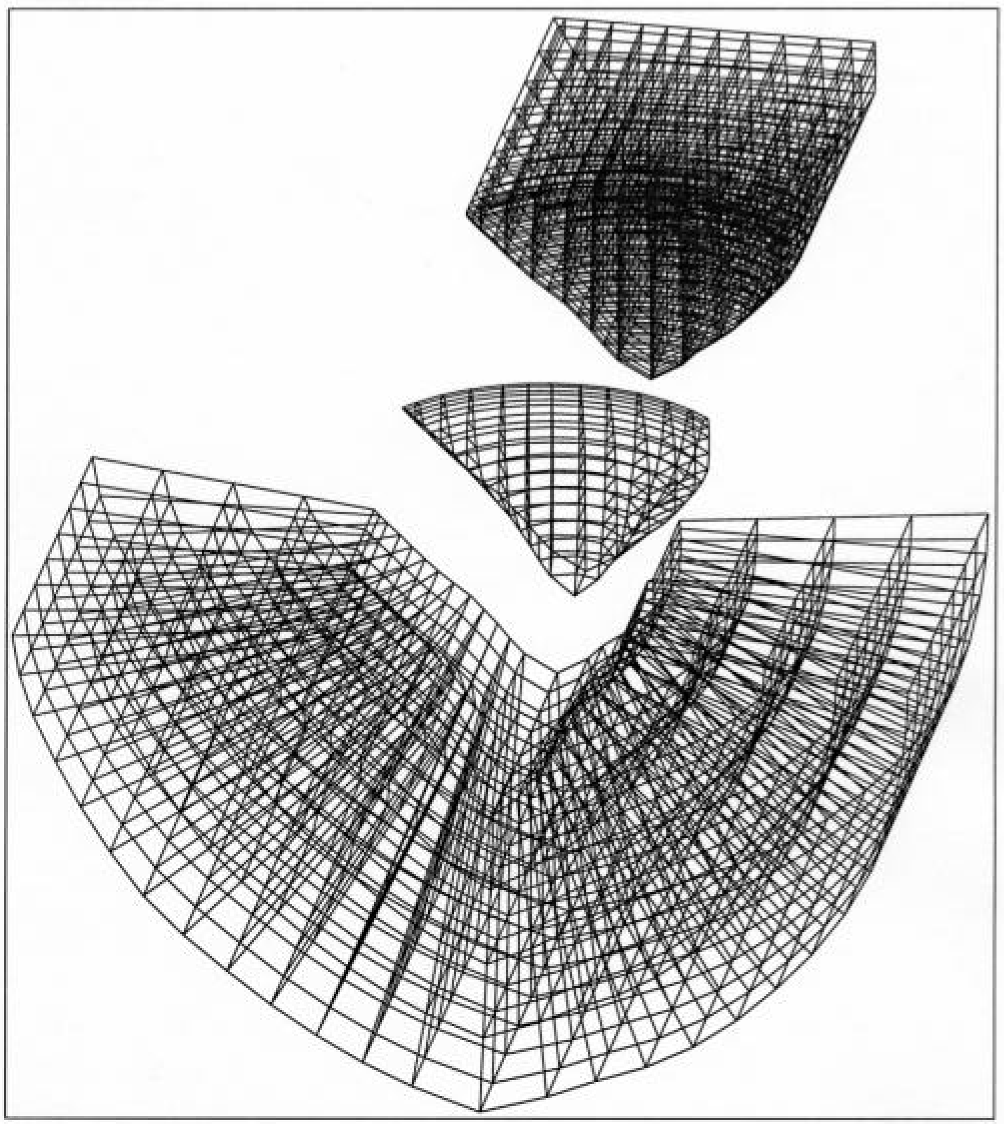

Figure 6. Finite element meshes for Pacoima Dam, water reservoir and rock foundation.

For the present study, SCADA was modified to accept nonuniform input. Earthquake loads are computed as those which when applied to the foundation mesh alone cause the interface to move at the desired motions. For the Pacoima Dam analysis, this requires that the recorded accelerograms at the abutments and base of the dam be interpolated to each dam node on the interface. Motions at nodes on the north side of the canyon are interpolated from the north abutment and base records, similar for the south side of the canyon using the south abutment and base records. Interpolation is performed channel by channel, and the interpolation at a node is weighted according to the elevation of the node. Before interpolation, any time shift is eliminated, and then the interpolated record is appropriately re-shifted based on its nodal elevation. For nodes located higher than the abutment stations, larger amplitudes and time shifts result. The interpolated motions, as a function of elevation, are applied to the water mesh as well.

Since channels 9 to 17 are located practically on the dam-rock interface, they should not be treated as free-field motions. That is, the dam nodes at the interface should be forced to move at the interpolated motions. To accomplish this, large stiffness terms were assembled into the foundation mesh for the translational degrees of freedom on the interface with the dam before the earthquake loads were computed. With this modification, the rock mesh provides flexibility only to the rotational degrees of freedom on the interface, as these are left free.

Material properties for the finite element model were chosen as follows: Young's moduli of 17,900 MPa (2600 ksi) for the concrete and 13,800 MPa (2000 ksi) for the rock, and unit weights of 23.6 kN/m^3 (150 lb/ft^3) for the concrete and 9.8 kN/m^3 (62.4 lb/ft^3) for the water. The rock modulus is in the range of a rather large variation of field data (Hall, 1988); it corresponds to a shear wave speed of about 1500 m/sec. The concrete modulus was selected to give natural frequencies close to those determined by MODEID. The frequencies resulting from this value, which is in the typical range for dam concrete, are 4.9hz for the first antisymmetric mode and 5.1 hz for the first symmetric mode. Also, based on the results of the system identification study, the damping level was set to 6% of critical.

Figure 7 shows a comparison between the recorded accelerations for channels 1, 2, 4 and 5 on the crest with ones computed by SCADA operating in the linear mode. The agreement is not particularly good, except for channel 4 where the motion is relatively small. The responses from the finite element model are too high for channels 1 and 2, and they contain a high frequency component at channel 5 not seen in the record (although it is present in the nearby station on the south abutment, see Figure 2). When displacements are compared (Figure 8), the agreement is better, although the response at channel 2 is still noticeably too high. In order to improve the performance of the finite element model, several modifications were tried: altering Young's modulus of concrete, raising the damping level, allowing full interaction between the dam and foundation rock, and permitting the contraction joints to open. None of these proved to be entirely successful; for example, increasing the damping to 12% of critical improved the agreement for channels 1 and 2 but worsened it for channel 5 (Figure 9).

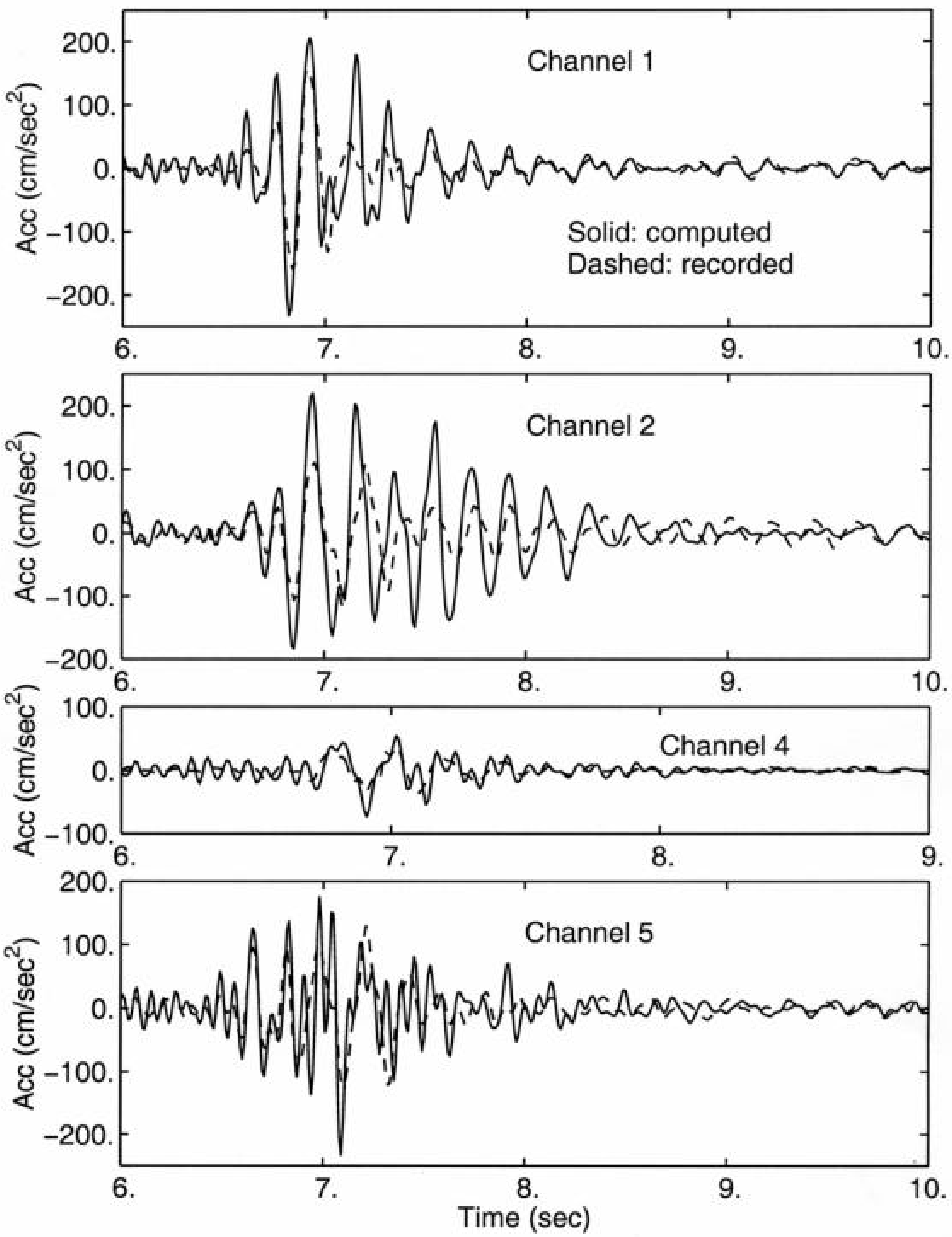

Figure 7. Comparison between the recorded accelerations at channels 1, 2, 4 and 5 and the computed accelerations from the SCADA finite element model with 6% damping.

Further work is underway to explain the differences between the recorded and computed responses of the dam. The first step of this effort is to examine in detail the behavior of the finite element model. Figure 10 shows computed displacement responses for channels 1, 2, 4 and 5 (same parameters as used for Figures 7 and 8) separated into the pseudo-static response and the dynamic responses from the first symmetric and

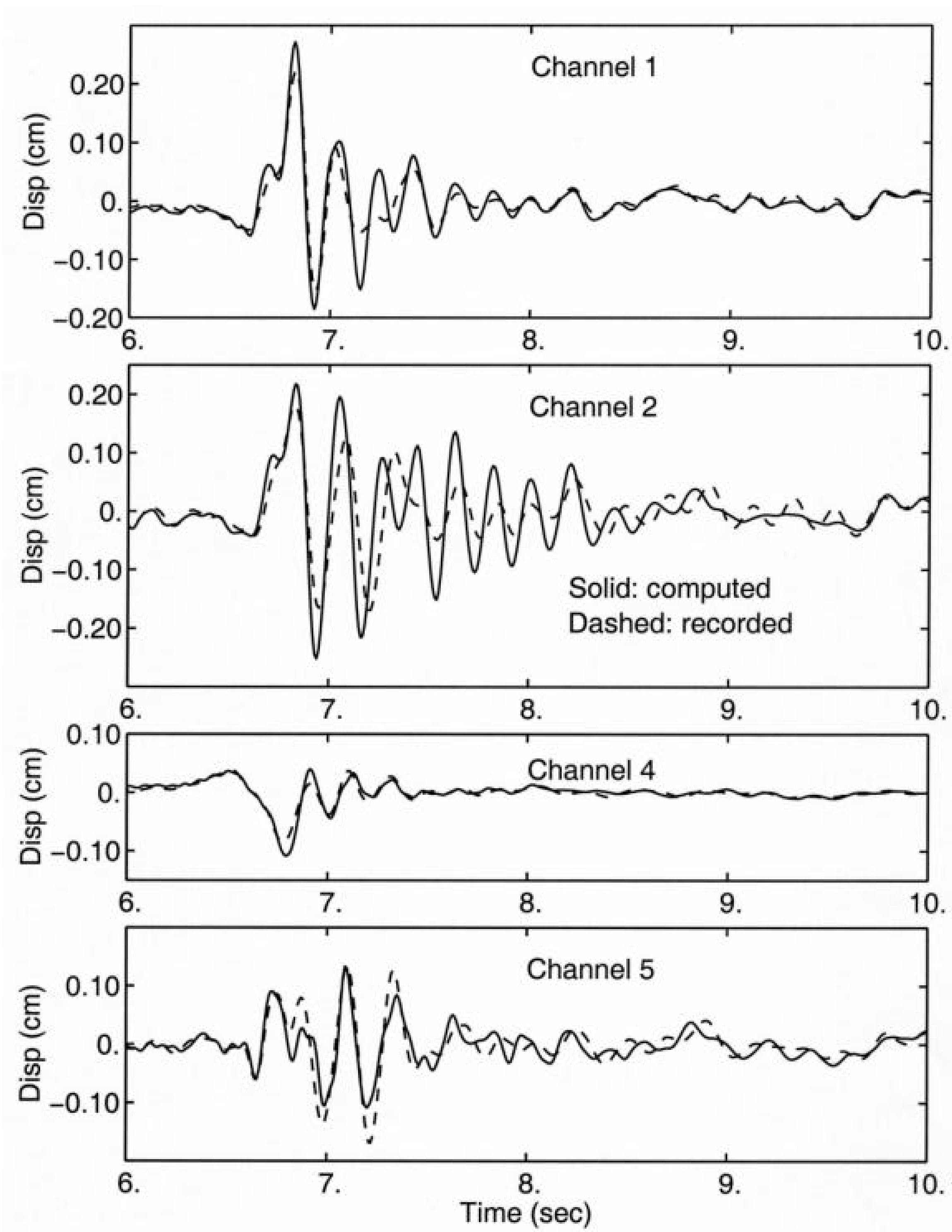

Figure 8. Comparison between the recorded displacement at channels 1, 2, 4, and 5 and the computed displacements from the SCADA finite element model with 6% damping.

antisymmetric modes. The largest contributions are made by the symmetric mode for channel 2, the antisymmetric mode for channels 1 and 5, and the pseudo-static response for channel 4; however, all three parts are important for each channel. It is expected that

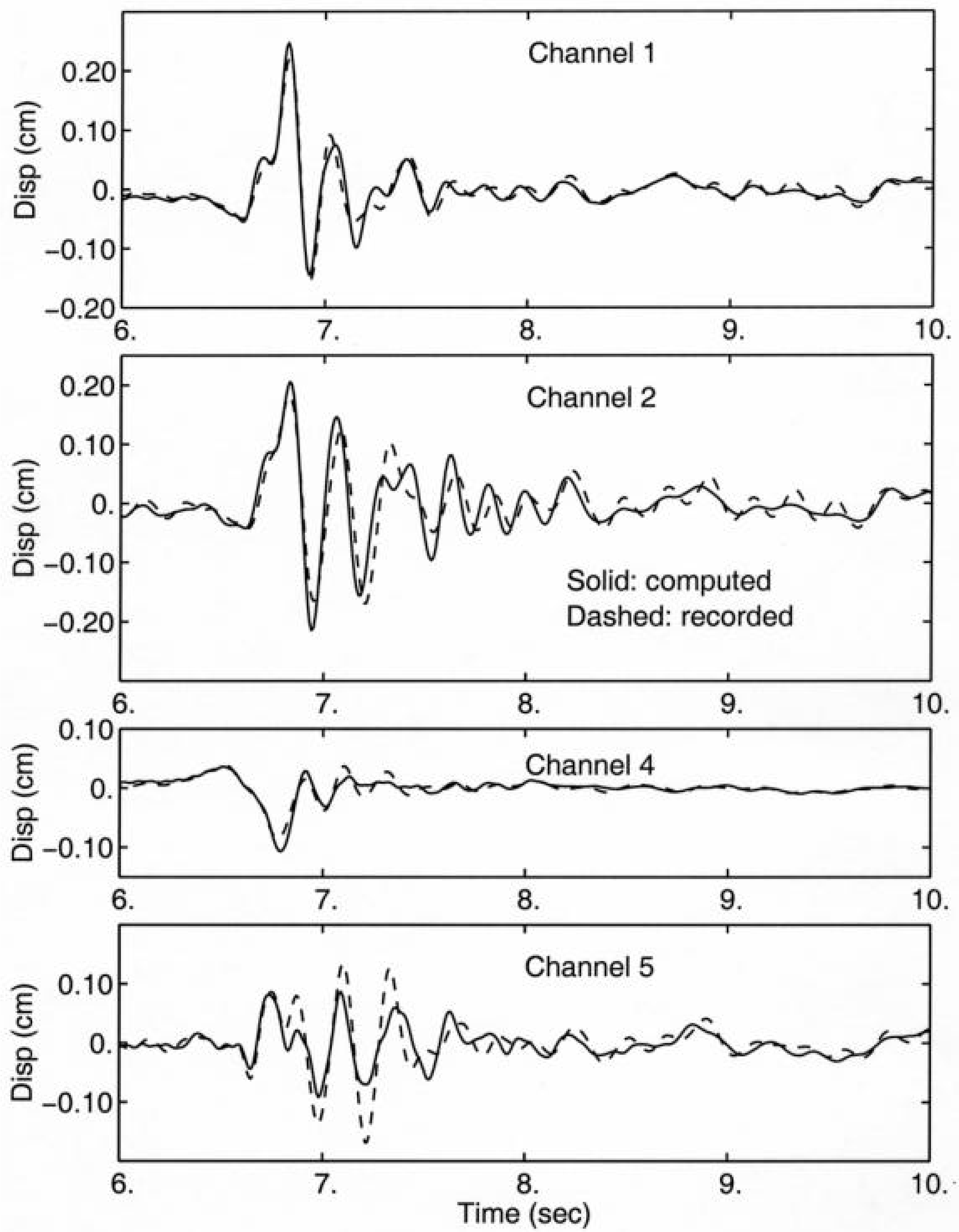

Figure 9. Comparison between the recorded displacements at channels 1, 2, 4 and 5 and the computed displacements from the SCADA finite element model with 12% damping.

the symmetric mode is excited mainly by ground motion in the upstream-downstream direction and the antisymmetric mode is excited mainly by cross-stream ground motion.

Plots of the two computed mode shapes at crest level appear in Figure 11, and the symmetric and antisymmetric features are evident. These shapes explain why the symmetric mode contributes more to the channel 2 response, and why the antisymmetric mode contributes more to the others. It is also evident that the presence of the antisymmetric mode in the channel 2 response, which is not insignificant, is sensitive to the location of this mode's cross-over point for radial motion (where the radial motion is zero). A small shift of the cross-over point toward the location of channel 2 would significantly reduce its contribution to the channel 2 response. This suggests that

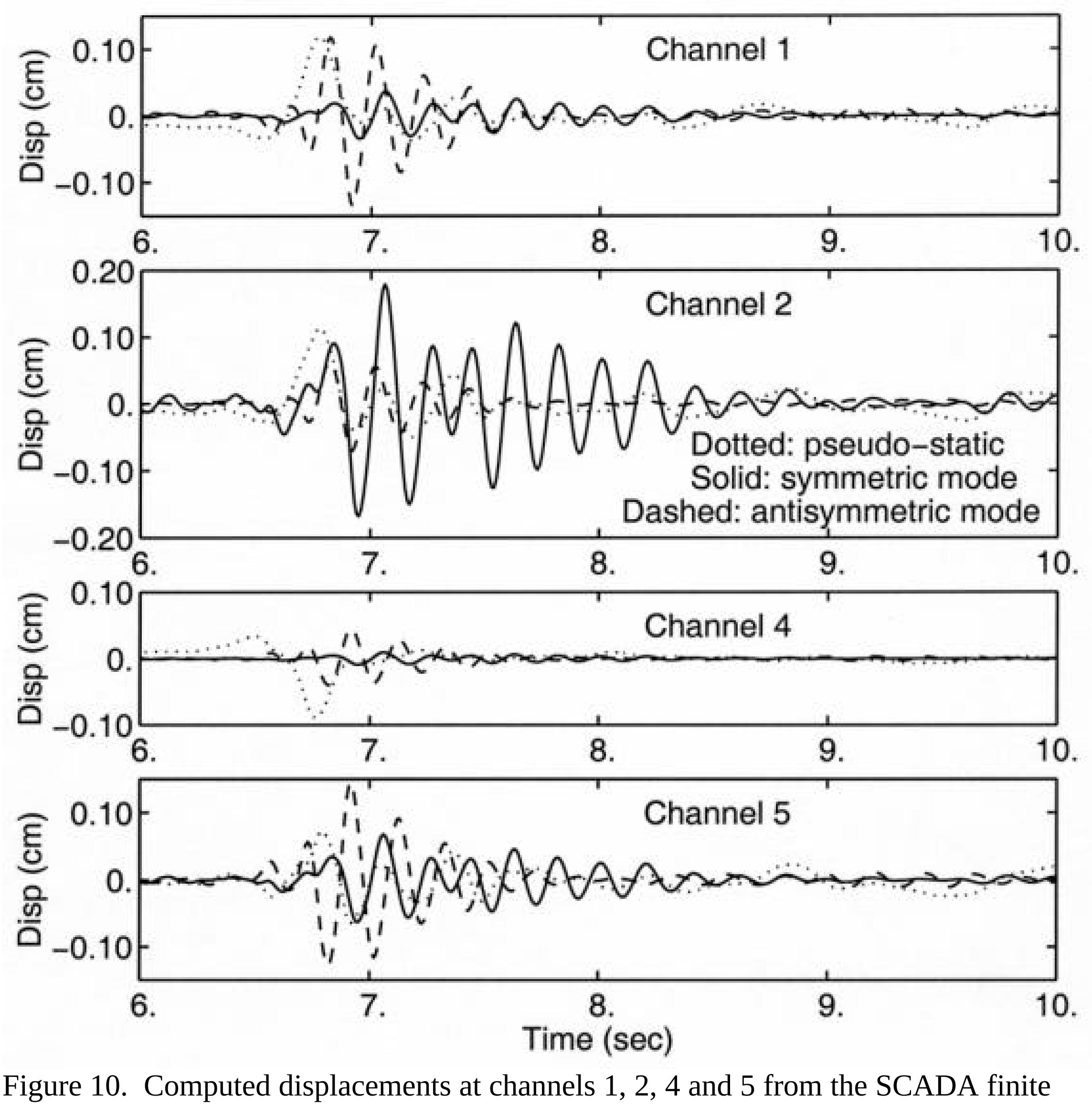

Figure 10. Computed displacements at channels 1, 2, 4 and 5 from the SCADA finite element model with 6% damping, separated into the pseudo-static response and the dynamic responses from the first two modes.

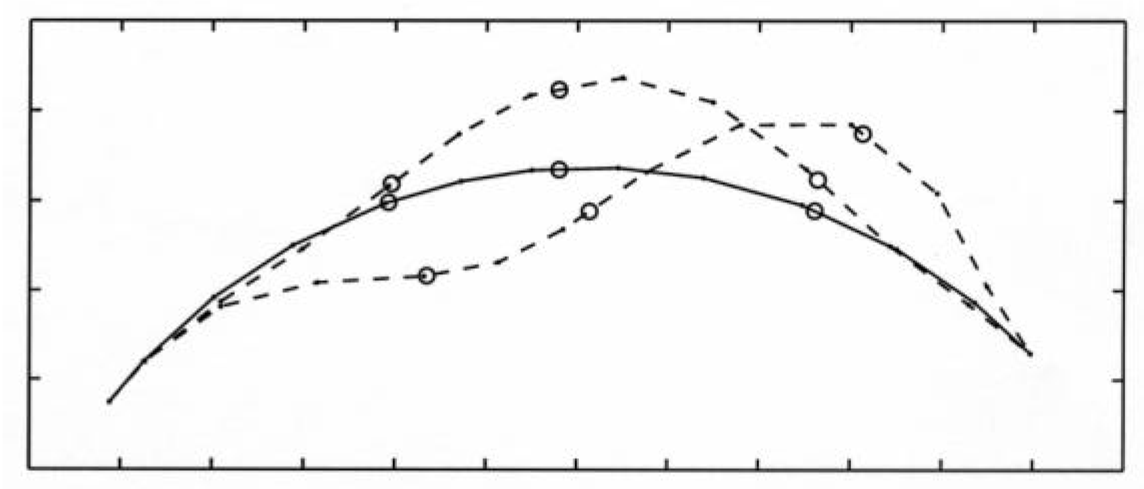

Figure 11. Plan view of the crest of the Pacoima Dam finite element model showing the fundamental symmetric and antisymmetric mode shapes (dashed lines). The open circles denote the three recording stations on the crest of the dam.

improvements to the finite element mesh, such as explicitly modeling the thrust block on the south abutment instead of treating it as part of the foundation rock, could be effective in obtaining better agreement to the recorded motions.

To be realistic, the ability of a finite element model to reproduce the recorded responses of an arch dam will always be less than that, say, for a building where uniform ground motion can often be assumed. Nonuniform ground motion can only be measured at a limited number of recording stations, meaning that a complete definition of the seismic input is subject to interpolation error. And the pseudo-static response associated with nonuniform ground motion involves deformation of the structure and so is different from point to point; whereas, the pseudo-static response to uniform ground motion is also uniform and identical to the ground motion itself. Finally, the presence of two modes which have nearly equal natural frequencies and which can contribute significant amplitude to the same line of motion over much of the structure is an additional complication not typically found in buildings.

Conclusions

For this progress report, some preliminary conclusions and suggestions for continuing work have been given in the preceding sections. As is apparent, there are several directions of research which can exploit the data obtained at Pacoima Dam on January 13, 2001. Of primary interest in these studies are the effects of nonuniform ground motion and the level of damping. In these respects, the data are proving to be extremely valuable even though they were generated by a minor earthquake of magnitude 4.3. Study of the records from the Northridge earthquake at Pacoima Dam as well as additional records obtained from future earthquakes will help to expand the conclusions reached by the present investigation.

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DISSEMINATION OF STRONG-MOTION DATA VIA THE INTERNET QUICK REPORT AND THE INTERNET DATA REPORT AT THE CISN ENGINEERING DATA CENTER

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Abstract

The Engineering Data Center (EDC) of the California Integrated Seismic Network (CISN) provides an Internet Quick Report (IQR) to distribute processed strong-motion data and detailed information on instrumented stations immediately after an earthquake. The coverage area of the IQR has been extended to statewide coverage and the Internet Data Report has been introduced. The EDC also has developed search functions to let users quickly access strong-motion data records from certain stations or certain structure types for different earthquakes based on station or structure characteristics.

Introduction

The California Integrated Seismic Network (CISN) is a newly formed consortium of institutions engaged in statewide earthquake monitoring. The five core members of CISN include the California Geological Survey (CGS, formerly the California Division of Mines and Geology), the California Institute of Technology, the Pasadena Office of the U.S. Geological Survey, the Menlo Park Office of the U.S. Geological Survey and the University of California, Berkeley. The California Office of Emergency Services (OES) serves as an ex-officio participant in the CISN.

The CISN has a southern California seismic data center at Pasadena, a northern California seismic data center in the Bay area, and the statewide Engineering Data Center. The CISN Engineering Data Center is operated by the CGS Strong Motion Instrumentation Program (CSMIP) in cooperation with the USGS National Strong Motion Program (NSMP). The EDC is currently at http://www.conservation.ca.gov/cisn-edc/ (Figure 1), but reflecting the dual-agency nature of the EDC, it will be operating in parallel at a USGS address later in 2002. This will provide the critically needed robustness and redundancy for the EDC, in the case of a major earthquake or other event at either location, which is a central goal of the CISN and of OES.

A primary goal of the EDC as well as the other two Data Centers is to provide rapid information products after an earthquake, ranging from the ShakeMap to distribution of the data and calculated parameters. A dedicated high-speed computer network, or Intranet, connecting all CISN partner agencies is currently under construction and standardized, consensus protocols for the exchange of earthquake parametric data and waveforms are being finalized by the CISN Standards Committee. With the completion of the Intranet, the CISN partner networks will routinely exchange strong-motion waveform and parameter data between centers and the EDC will assemble strong-motion data sets for the earthquake engineering community incorporating data from all CISN stations.

The EDC uses the Internet Quick Report (IQR) to rapidly disseminate strong-motion data for engineering applications right after major earthquakes. The IQR is based on the concept of the traditional Quick Report, streamlined for automated generation. A total of six IQRs have been released after the earthquakes of M~4 and larger since August 2001. In addition, a search function has been developed to provide users a simple but versatile tool to locate strong-motion data of specific interest during ongoing searches and studies between earthquakes. The design of the search function effectively reorganizes access to the strong-motion data at the EDC according to the parameters of the instrumented station or structure. This provides two essentially orthogonal paths to request data, by earthquake for all stations or by station/structure type for all earthquakes, which allows users to quickly locate the data of interest for their engineering applications.

The Internet Quick Report

An Internet Quick Report is generally prepared for earthquakes over magnitude 4.0, for which a ShakeMap is also released by CISN. The content of the IQR is dynamic and cumulative after an earthquake, building as new data continues to be recovered. Initial work on the IQR is described in Shakal and Scrivner (2000) and Lin et al. (2001). An example of the current IQR, from a recent earthquake in southern California, is shown in Figure 2. It lists recovered data from the CSMIP and NSMP networks, in order of increasing epicentral distance. At the top of the IQR web page is given the name and date of the earthquake, links to related information about the event at other CISN sites (location, magnitude and ShakeMap), and the time of last modification of the table. The table lists peak acceleration values and station distances for the strong-motion records recovered. Each row of the table represents one record and includes the station name and number, network, epicentral distance, and peak horizontal acceleration, on the ground and the structure. The row also includes buttons for viewing and/or downloading the data once the data itself is available at the EDC. Information regarding the station or instrumented structure is accessible directly using the Internet link under the station name.

The table shown, designed for viewing using Internet browsers, is complemented by a more comprehensive table available as an ASCII text file using a link at the top of the page. This table can easily be imported into a spreadsheet program for analyses by a user. Both the web table and the text table have a date-time stamp to indicate when it was last modified by update or addition of data.

The Internet Data Report for Previous Earthquakes

The discussion above is focused on data in the immediate post-earthquake period. Earthquake data is also important for long-term analysis, beyond the post-earthquake response time frame. In the past, paper-copy Quick Reports (e.g., CSMIP, 1994) were the pre-Internet analog to the Internet Quick Report. But those Quick Reports were followed by a final hard-copy report on the event's strong motion data, usually released in one month after the event (e.g., Porcella et al., 1994; Shakal et al., 1994). In many ways the IQR is as comprehensive as that report, so one could say that, using new technology, a product is being produced in 30 minutes very comparable to what used to be available only after 30 days. To parallel the final hard copy data report, the Internet Quick Report for an event will become the Internet Data Report, to reflect its more final nature, after enough time has gone by for all data (including that from analog instruments) to be included and quality controlled.

The EDC is creating the Internet Data Reports for all significant previous earthquakes to provide users the same access to previous strong-motion data, including structural data, as for new data. An example showing part of the Internet Data Report for the 1994 Northridge earthquake is shown in Figure 3. The Internet Data Reports have the same format as the IQR, though they will be processed manually in part, and placed in the archive section of the EDC, paralleling the way the CISN ShakeMaps are archived.

Searching for Strong-Motion Data at the CISN Engineering Data Center

The above sections considered access to data on a single-earthquake basis. A second path for access to strong-motion data is to use the search function of the EDC. The data contents for the search function are updated immediately following an IQR update operation as part of the EDC process. Contrary to the IQR and IDR, in which data for a single earthquake is listed for all stations, the search function lists data for a single station (or class of stations) for various earthquakes. The user can access the strong-motion data for a structure of interest (a certain building, bridge, or dam) or for a class of structures (e.g., all mid-height steel buildings) by specifying the categorization or type of the structure.

The design goal of the EDC search function is to distribute strong-motion data from stations of the CISN network in a rapid manner and also provide structural data and associated station information. As such, it complements the extensive database at the COSMOS virtual data center, which includes ground-response records from stations around the world and comprehensive search capabilities (Archuleta, 2000). It also complements the newly completed Internet Site for European Strong Motion Data (ISESD), which provides access to records and station information for data recovered in countries of Europe and the Middle East (Ambraseys, 2000).

The layout of the EDC search function is a typical top-down tier approach that guides the user through a series of choices. The user can further confine the search criteria by entering keywords in appropriate fields anytime during the search process. Results of a given search are presented in a table listing all stations that matched the search criteria. Each station has a direct link in the result table that leads the user to a full list of readily accessed strong-motion data for the station.

The initial search page, which includes six predefined station categories (ground response, buildings, bridges, dams, geotechnical arrays, and other structures) is shown in Figure 4. At the first level of the search process, the user starts a search request by selecting a station category. The system responds by displaying second level options for the selected category. For ground response stations, the user can search for strong-motion data based on station name, station number, and site geology. For structural stations, the second level allows search options that are unique to the structure category as well as the same search criteria as for ground response stations. An example of search options for a building station is shown in Figure 5. Similar structure-type specific options are included for bridges and for dam stations. There is also a third level search option for building stations that considers the lateral force resisting system used in the structural design.

A data search request results in a table listing all stations that match the search criteria. Figure 6 shows an example of a search result for high-rise (8-stories and over) steel frame buildings (the building classifications adopted are those of FEMA 310). Within the table, there is a link for each station, which will take the user to a web page with a list or records just for that station (e.g., Figure 7). The station-data page is again a peak acceleration table. Each row of the table represents one record, and includes the name of the earthquake (which incorporates a link to the Internet Data Report for that event), the epicentral distance, and the peak acceleration on the ground and on the structure. The strong-motion data records for the earthquake can be directly viewed or downloaded via the web link buttons.

Continuing Development

As the infrastructure of CISN develops it will include a dedicated high-speed T1 data network ring, or Intranet, for rapid and robust post-earthquake data exchange among the CISN networks.   The level of data exchange for strong-motion waveform and parameter data among CISN partner agencies will increase greatly when this network is operational later in 2002. At that time the IQR process of the EDC will serve strong-motion data from all CISN agencies in a fully automatic mode.

To provide users ready access to strong-motion data from previous earthquakes, Internet Data Reports are being created. The completion of this effort for those earthquakes, coupled with the station data pages for all the structures in the USGS and CGS strong motion networks, will make the CISN engineering data center fully effective, supplanting traditional manual means of accessing and providing strong motion data and station information.

Summary

The CISN Engineering Data Center, operated by the federal-state partnership of the CSMIP of the California Geological Survey and the National Strong Motion Program of the US Geological Survey, will greatly accelerate access to data after events and allow users to conveniently obtain data for specific structures and structure types. The development of the EDC is continuing and is summarized below:

- The CISN Engineering Data Center can be accessed at www.conservation.ca.gov/cisn-edc, and will be available, in parallel, at a usgs.gov address in 2002.

- The EDC has developed the Internet Quick Report which will be available automatically within 15 minutes or less after M>4 earthquakes by later in 2002; until then it will be partly manual and available within minutes to hours of significant earthquakes.

- A search function to allow users to conveniently access strong-motion earthquake data and detailed information on instrumented structures and other stations, including location, site geology, structural system, sensor layouts, and other information has been developed.

- The EDC is populating its data archives to include strong-motion data and station/structure information from previous earthquakes, from the CSMIP and NSMP networks and the other partners.

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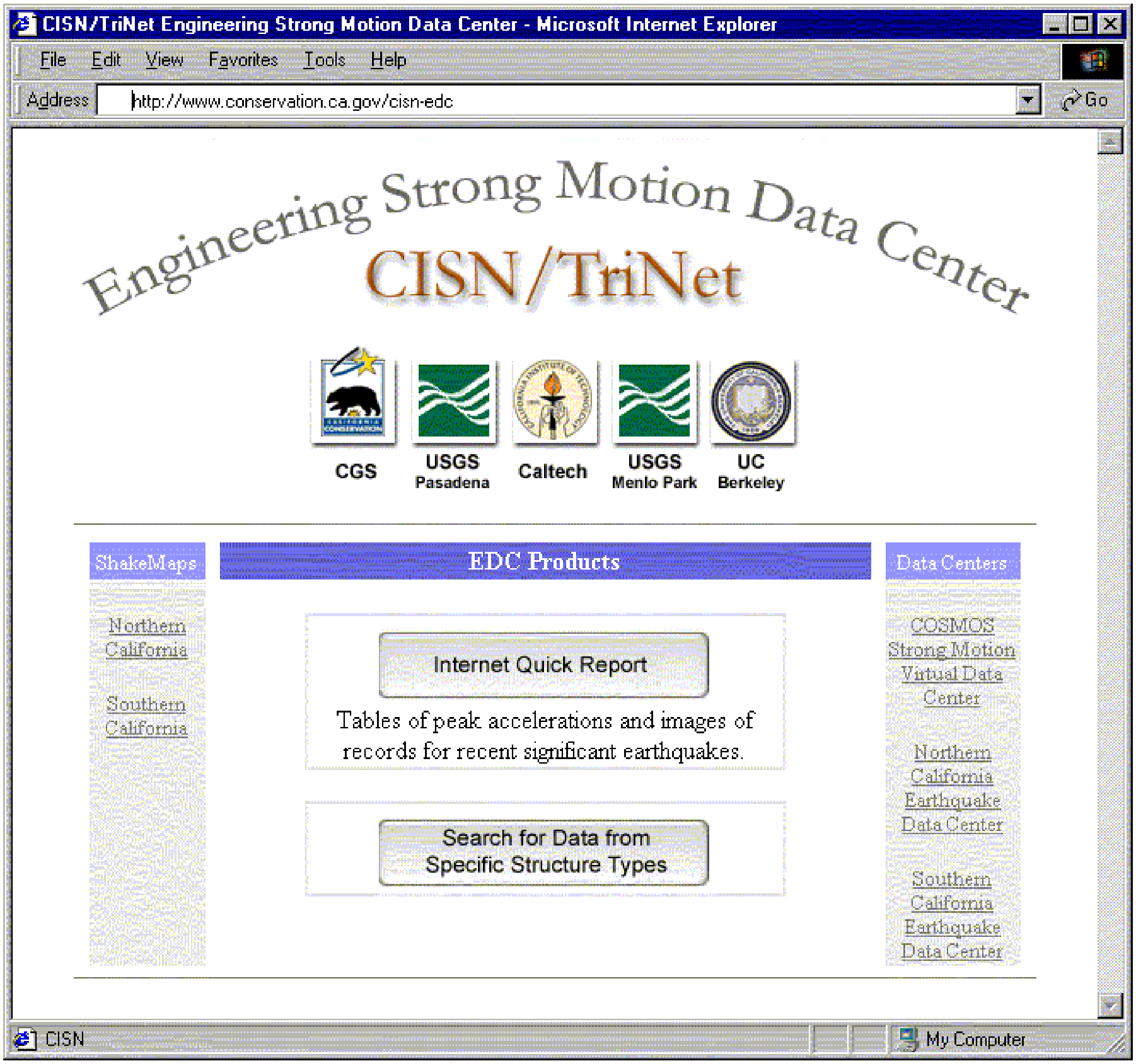
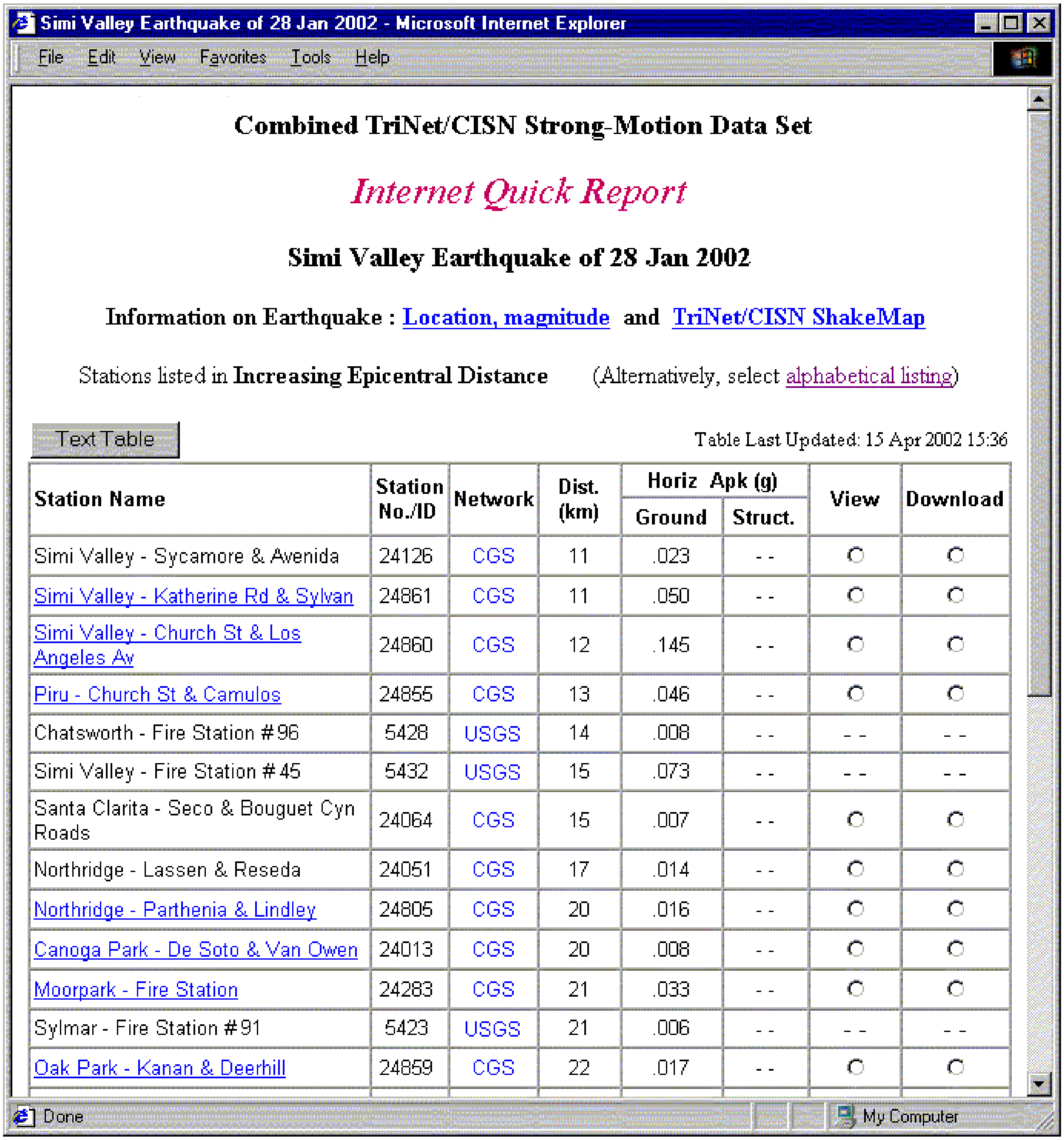


Figure 1. The home page of the CISN Engineering Strong Motion Data Center, with links to Internet Quick Reports, data searches by station/structure type, and links to the other two CISN data centers, the COSMOS data center, and the ShakeMaps for northern and southern California.



Simi Valley Earthquake of 28 Jan 2002 - Microsoft Internet Explorer

File Edit View Favorites Tools Help

Combined TriNet/CISN Strong-Motion Data Set

Internet Quick Report

Simi Valley Earthquake of 28 Jan 2002

Information on Earthquake : Location, magnitude and TriNet/CISN ShakeMap

Stations listed in **Increasing Epicentral Distance** (Alternatively, select alphabetical listing)

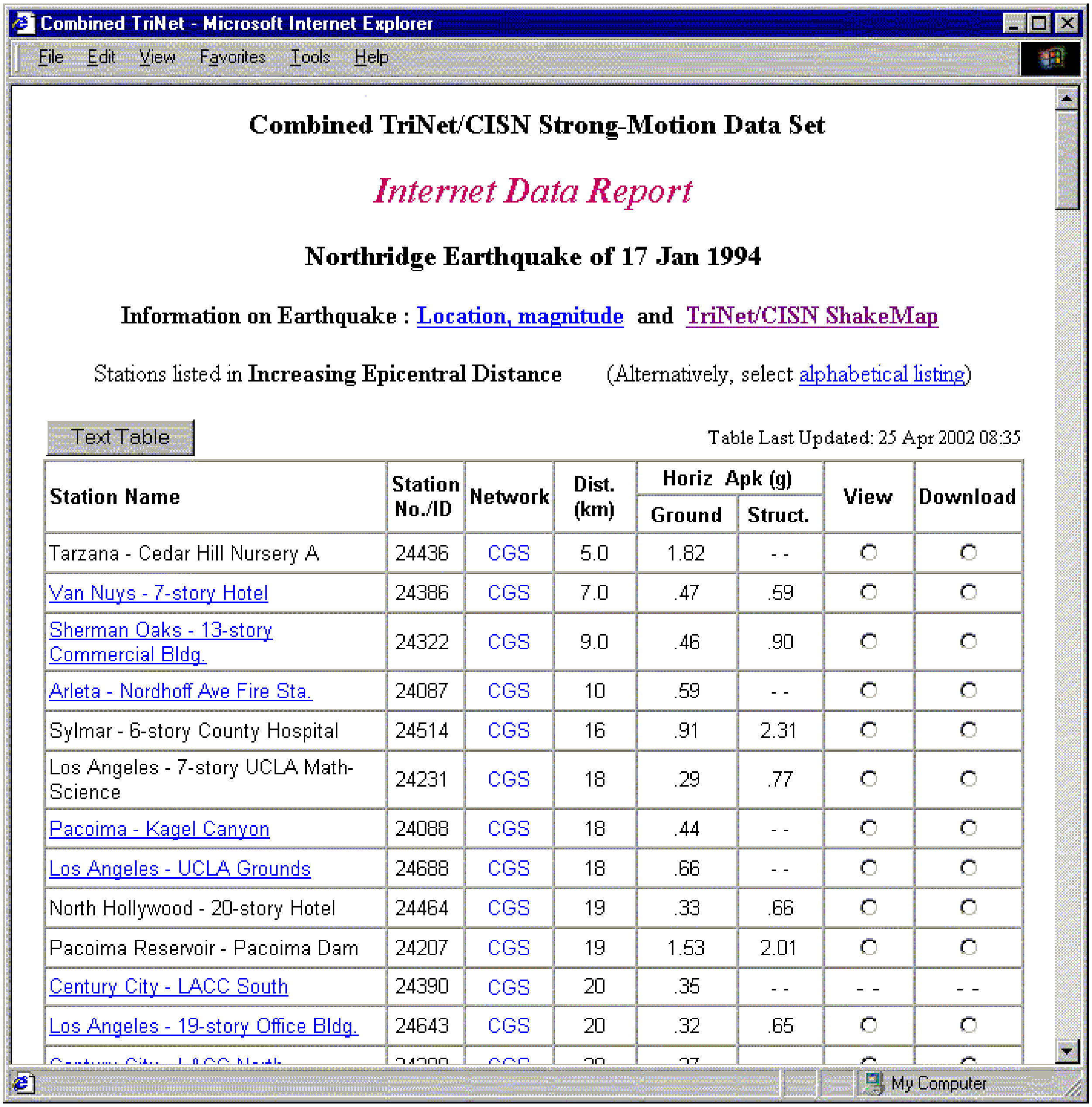
Text Table

Table Last Updated: 15 Apr 2002 15:36

Station Name	Station No./ID	Network	Dist. (km)	Horiz Apk (g)		View	Download
				Ground	Struct.		
Simi Valley - Sycamore & Avenida	24126	CGS	11	.023	- -	○	○
Simi Valley - Katherine Rd & Sylvan	24861	CGS	11	.050	- -	○	○
Simi Valley - Church St & Los Angeles Av	24860	CGS	12	.145	- -	○	○
Piru - Church St & Camulos	24855	CGS	13	.046	- -	○	○
Chatsworth - Fire Station #96	5428	USGS	14	.008	- -	- -	- -
Simi Valley - Fire Station # 45	5432	USGS	15	.073	- -	- -	- -
Santa Clarita - Seco & Bouguet Cyn Roads	24064	CGS	15	.007	- -	○	○
Northridge - Lassen & Reseda	24051	CGS	17	.014	- -	○	○
Northridge - Parthenia & Lindley	24805	CGS	20	.016	- -	○	○
Canoga Park - De Soto & Van Owen	24013	CGS	20	.008	- -	○	○
Moorpark - Fire Station	24283	CGS	21	.033	- -	○	○
Sylmar - Fire Station #91	5423	USGS	21	.006	- -	- -	- -
Oak Park - Kanan & Deerhill	24859	CGS	22	.017	- -	○	○

Done My Computer

Figure 2. An example Internet Quick Report for the M4.2 earthquake that occurred near Simi Valley, CA on Jan. 28, 2002. The table is sorted in epicentral distance order; alphabetical order can be selected at the top, and an ASCII table can be downloaded for spreadsheet or other analysis using the ‘Text Table’ link. The stations for which strong-motion data is available for viewing and/or downloading are indicated by the presence of push buttons in the right columns. For underlined stations, a linked page may be accessed which contains station photographs and site information.



Combined TriNet/CISN Strong-Motion Data Set

Internet Data Report

Northridge Earthquake of 17 Jan 1994

Information on Earthquake : Location, magnitude and TriNet/CISN ShakeMap

Stations listed in **Increasing Epicentral Distance** (Alternatively, select alphabetical listing)

Text Table

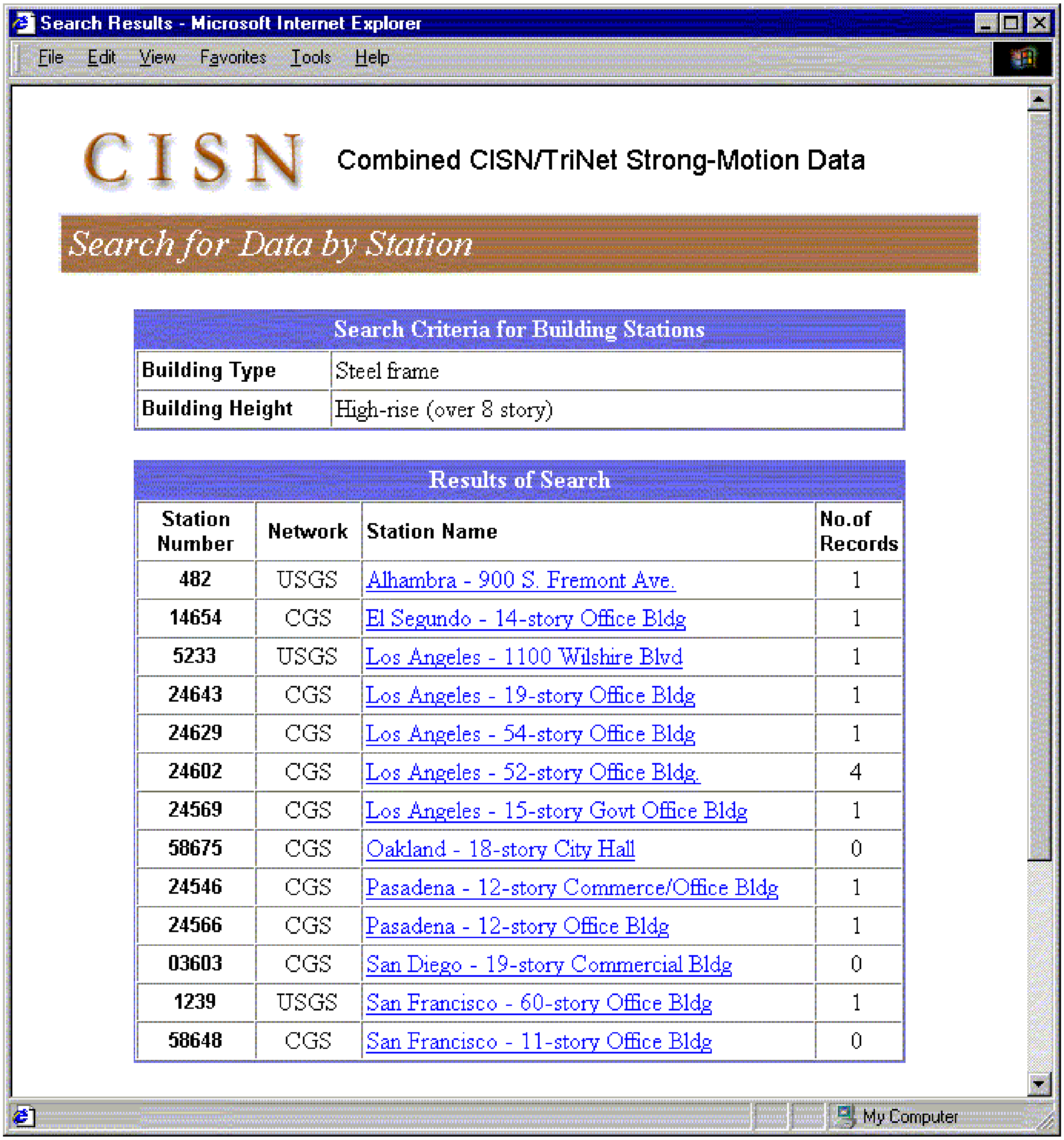
Table Last Updated: 25 Apr 2002 08:35

Station Name	Station No./ID	Network	Dist. (km)	Horiz Apk (g)		View	Download
				Ground	Struct.		
Tarzana - Cedar Hill Nursery A	24436	CGS	5.0	1.82	- -	○	○
Van Nuys - 7-story Hotel	24386	CGS	7.0	.47	.59	○	○
Sherman Oaks - 13-story Commercial Bldg.	24322	CGS	9.0	.46	.90	○	○
Arleta - Nordhoff Ave Fire Sta.	24087	CGS	10	.59	- -	○	○
Sylmar - 6-story County Hospital	24514	CGS	16	.91	2.31	○	○
Los Angeles - 7-story UCLA Math-Science	24231	CGS	18	.29	.77	○	○
Pacoima - Kagel Canyon	24088	CGS	18	.44	- -	○	○
Los Angeles - UCLA Grounds	24688	CGS	18	.66	- -	○	○
North Hollywood - 20-story Hotel	24464	CGS	19	.33	.66	○	○
Pacoima Reservoir - Pacoima Dam	24207	CGS	19	1.53	2.01	○	○
Century City - LACC South	24390	CGS	20	.35	- -	- -	- -
Los Angeles - 19-story Office Bldg.	24643	CGS	20	.32	.65	○	○

Figure 3. An example of the Internet Data Report table, sorted in epicentral distance order, for the M6.7 Northridge earthquake of Jan. 17, 1994. The table parallels the functionality of the Internet Quick Report, but is permanently available, beyond the time of post-earthquake response, and includes records from analog film stations and other records that may be recovered manually (this is an example and only includes CGS data).

Figure 4. The initial page of the EDC search function. The station data are categorized in six major station types including ground response, buildings, bridges, dams, geotechnical arrays, and other structures.

Figure 5. An example of the second level search options for building stations. Shown in the figure are the pull-down menu choices for building construction material. A following selection menu can be used to establish the desired building height category, and, if desired, the local geology at a structure.


Search Results - Microsoft Internet Explorer

File Edit View Favorites Tools Help

CISN Combined CISN/TriNet Strong-Motion Data

Search for Data by Station

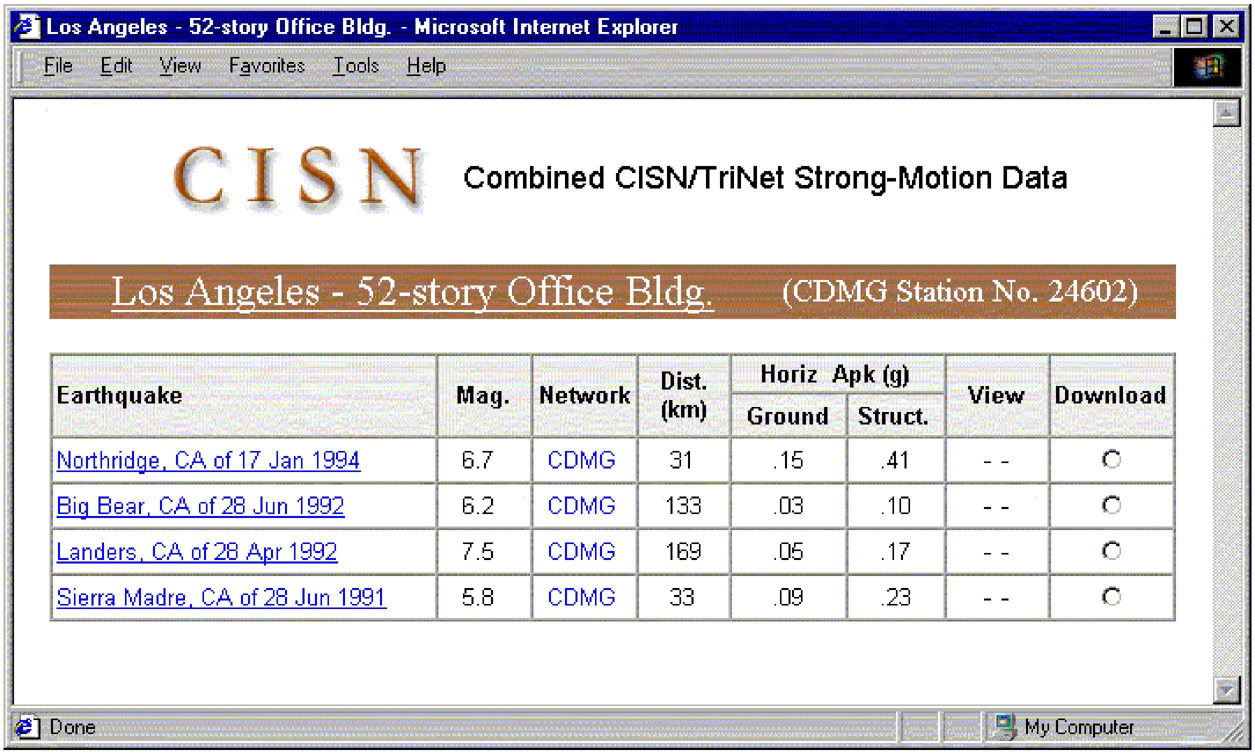
Search Criteria for Building Stations	
Building Type	Steel frame
Building Height	High-rise (over 8 story)

Results of Search

Station Number	Network	Station Name	No.of Records
482	USGS	Alhambra - 900 S. Fremont Ave.	1
14654	CGS	El Segundo - 14-story Office Bldg	1
5233	USGS	Los Angeles - 1100 Wilshire Blvd	1
24643	CGS	Los Angeles - 19-story Office Bldg	1
24629	CGS	Los Angeles - 54-story Office Bldg	1
24602	CGS	Los Angeles - 52-story Office Bldg.	4
24569	CGS	Los Angeles - 15-story Govt Office Bldg	1
58675	CGS	Oakland - 18-story City Hall	0
24546	CGS	Pasadena - 12-story Commerce/Office Bldg	1
24566	CGS	Pasadena - 12-story Office Bldg	1
03603	CGS	San Diego - 19-story Commercial Bldg	0
1239	USGS	San Francisco - 60-story Office Bldg	1
58648	CGS	San Francisco - 11-story Office Bldg	0

My Computer

Figure 6. An example of the result of a search request for data from high-rise (8-stories and over) steel frame buildings. The table indicates the network that responsible for the instrumented building and the number of records available (more records will be added as the existing structural records and information is added to the archive). A user can click on any of listed stations to link to a station-data web page.



Earthquake	Mag.	Network	Dist. (km)	Horiz Apk (g)		View	Download
				Ground	Struct.		
Northridge, CA of 17 Jan 1994	6.7	CDMG	31	.15	.41	- -	○
Big Bear, CA of 28 Jun 1992	6.2	CDMG	133	.03	.10	- -	○
Landers, CA of 28 Apr 1992	7.5	CDMG	169	.05	.17	- -	○
Sierra Madre, CA of 28 Jun 1991	5.8	CDMG	33	.09	.23	- -	○

Figure 7. An example of the station-data web page for a specific station selected in Figure 6. The table shows the collection of strong-motion records for different earthquakes that may be selected from this specific station. A user can click on any of these links to access the IQR or the Internet Data Reports for the earthquakes.

DEVELOPMENT OF AN ENGINEERING MODEL OF THE AMPLITUDE AND DURATION EFFECTS OF BASIN GENERATED SURFACE WAVES

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Abstract

Basin waves are polarized predominantly in the directions parallel to and normal to the edge of the basin, consistent with the assumption made by Joyner (2000). There are significant differences between the amplitudes of the horizontal component parallel to the basin edge, which is predominantly Love waves, and the horizontal component perpendicular to the basin edge, which is predominantly Rayleigh waves, although these amplitudes will be affected by the relative strength of the incoming waves, which depends on focal mechanism and other factors. This is also consistent with the assumption made by Joyner (2000). There is a significant dependence of the ground motion amplitude and duration on basin depth. Basin depth dependence was not included in the Joyner (2000) model. When depth increases away from the basin edge (i.e. when basin depth and basin edge distance are correlated), this can cause ground motion amplitudes and durations to increase away from the basin edge. The Husid plot derived from the velocity waveform provides an appropriate duration measure that is independent of the absolute amplitude level.

Introduction

Joyner (2000) developed a procedure for modifying standard spectral attenuation relations to account for the amplitude effects of surface waves in deep sedimentary basins. The objective of this project is to extend his work so that it is more broadly applicable in earthquake engineering. We are extending the model to include duration in addition to spectral amplitudes. We are extending the model to include basins other than the Los Angeles basin. In particular, we are including data from shallower basins, such as the San Bernardino, San Fernando, Santa Clara, and Eel River basins, in which the basin effects are expected to extend to shorter periods. We are extending the lower bound of the period range covered by Joyner from 3 seconds to 1 second, which will make the model relevant to a much larger number of structures. The result of this study will be a model, suitable for earthquake engineering application, that modifies standard ground motion models to account for the amplitude and duration effects of basin generated surface waves. This paper describes results that have been obtained to date.

Mode of Generation of Basin Waves

The mode of generation of basin-trapped waves is illustrated schematically in Figure 1. If a seismic wave enters a basin through its edge, it can become trapped within the basin if post-critical incidence angles develop. The resulting total internal reflection at the base of the layer is illustrated at the top right of Figure 1. In the lower part of Figure 1, simple calculations of the basin response are compared with those for the simple horizontal layered model shown on the left side of the figure. In each case, a plane wave is incident at an inclined angle from below. The left side of the figure shows the amplification due to impedance contrast effects that occurs on a flat soil layer overlying rock (bottom) relative to the rock response (top). A similar amplification effect is shown for the basin case on the right side of the figure. However, in addition to this amplification, the body wave entering the edge of the basin becomes trapped, generating a surface wave that propagates across the basin. Current empirical ground motion attenuation relations do not distinguish between sites located inside and outside basins, and tend to underestimate the ground motions recorded in basins.

Joyner Model of Basin Effects

The basic concept underlying the Joyner (2000) model is that ground motions from an earthquake occurring outside a basin attenuate normally until they reach the basin edge, and then attenuate at a slower rate after entering the basin. The ground motion model is given by the equation:

$$\mathrm{Log}\ y = f(M, R_E) + c + b\, R_B$$

In this equation, y is the response spectral amplitude, $f(M, R_E)$ is an attenuation relation for non-basin conditions, M is moment magnitude, R_E is the distance from the earthquake to the basin edge, and R_B is the distance from the basin edge to the recording site. The parameter c is a measure of the coupling between the incident body waves and the surface waves in the basin, and the parameter b controls the attenuation with distance in the basin. Parameters b and c are period dependent parameters that are derived from the data. We are extending this model to include the effect of the depth H to crystalline bedrock below the recording site:

$$\mathrm{Log}\ y = f(M, R_E) + c + b\, R_B + a\, H$$

The Joyner model was developed for three components: parallel to the basin edge, perpendicular to the basin edge, and vertical. The perpendicular attenuation is found to be lower than the parallel attenuation. Joyner attributes this to the lower attenuation of Rayleigh waves (on the perpendicular component) than Love waves (on the parallel component) due to differences in the Q (damping) of P and S waves. However, unless the ray path is normal to the basin edge, this simple partitioning of wave types does not hold, because the site-to-source azimuth and the strike of the basin edge are independent variables. The Rayleigh waves should be on the radial component and the Love waves should be on the transverse component. For earthquakes located north of the Los Angeles basin, Joyner's assumption is reasonably valid, but for the Landers, Big Bear and Hector Mine earthquakes, the Love waves are closer to being perpendicular to the basin edge than parallel. However, it is possible that lateral refraction of the surface waves at the edge of the basin may tend to orient the waves in the directions assumed by Joyner.

Analysis of Polarization of Basin Waves

In Joyner's model, differences are recognized between the basin edge parallel and basin edge normal components. This model is based on the expectation that there is lateral refraction of surface waves at the basin edge. Alternative models could be based on the radial and tangential components, or the average horizontal component. Our first goal is to determine whether the data are consistent with Joyner's assumption.

We illustrate our analysis using the recordings of the 1999 Hector Mine earthquake in the San Bernardino basin. Figure 2 is a map showing the three fault segments of the Hector Mine earthquake and the locations of strong motion recording stations. The small shaded box contains stations located in and near the San Bernardino basin, shown in more detail in Figures 3 and 4. Profile A-A' in Figures 3 and 4 crosses the San Bernardino basin at right angles to the San Andreas fault, which forms the northeastern boundary of the basin. The San Jacinto fault forms the southwestern boundary of the basin. The basin structure is shown by the depth contours to bedrock (in km) and by the cross sections in Figure 4. The basin gradually thickens away from the San Andreas fault, and reaches a maximum depth of about 1 km near the San Jacinto fault, which is associated with a marked step in basement topography.

The lowpass filtered velocity waveforms of the Hector Mine earthquake recorded in and near the San Bernardino basin along profile A-A' are shown in Figure 5. At the top of Figure 5, we show the tangential (N154E) component on the left and the radial (N244E) component on the right. Because of the orientation and strike-slip mechanism of the Hector Mine earthquake, the radial direction is nodal, and nearly all of the energy is on the orthogonal tangential component. This can be seen in the waveforms of the closest recording station 5331 at the top of Figure 5, which lies just north of the San Andreas fault, outside the basin. There are large SH and Love waves on the N154E tangential component because they are near the maximum in the radiation pattern, and small Rayleigh waves on the radial N244E component because they are almost nodal.

At the bottom of Figure 5, we show the basin edge parallel (N310) component on the left and the basin edge normal (N220E) component on the right. Comparing these profiles with those at the top of Figure 5, we can see a clear change in the polarization of the ground motion. At basin stations (distances between 7 and 12 km from the San Andreas fault), the large SH and Love waves are much better separated from the nodal Rayleigh waves in the basin edge orientation shown at the bottom of Figure 5 than in the radial and tangential orientation shown at the top of Figure 5. In contrast, for station 5331 described above, the separation between these wave types for the basin orientation is degraded as expected, because this station is outside the basin.

Additional support for this interpretation of the polarization of the ground motions in the basin edge normal and basin edge parallel directions comes from polarization analysis using the method of Vidale (1986). Figure 6 shows the results of this analysis for the recording of the Hector Mine earthquake at station sbmv, located within the basin. The analysis on the left side of the figure is for the radial and tangential directions, and the analysis on the right side of the figure is for the basin edge normal and basin edge parallel directions. The latter orientation gives strike angles closer to 90 degrees, representing Love waves with particle motion parallel to the basin edge.

The data analyses shown in Figures 5 and 6 provide clear evidence of lateral refraction of surface waves at the basin edge. We conclude that basin waves are polarized predominantly in the directions parallel to and normal to the edge of the basin. This is consistent with the assumption made by Joyner (2000).

Generally, lateral refraction will cause SH and Love waves to be preferentially oriented on the basin edge parallel component, and Rayleigh waves to be oriented on the basin edge normal component, as illustrated in the Hector Mine recordings shown in Figures 5 and 6. Consequently, we expect these two components to have different amplitudes and to attenuate differently with distance away from the basin edge. For the Hector Mine recordings, radiation pattern effects caused a large difference in amplitude between these two components. These observations justify the use of separate attenuation functions for the basin edge normal and basin edge parallel components in the Joyner (2000) model. In that model, the basin edge parallel component attenuates more rapidly than the basin edge normal component.

Arias Intensity and Duration of Basin Waves

Since our basin ground motion model will include both amplitude and duration parameters, we plan to use a definition of duration that does not depend on the absolute level of the ground motion. We have evaluated the effectiveness of the Husid duration (Husid, 1969) of the velocity time history, which is used in the Abrahamson and Silva (1997b) model, as a measure of the duration of surface waves.

The method that we use to measure the Arias intensity and duration of basin waves is illustrated in Figure 7. This figure shows the recorded velocity time histories of the radial, tangential, radial, basin edge parallel, basin edge normal components, and the cumulative square of each time history (Husid, 1969), which represents the energy. The Arias intensity is defined as the total value of the cumulative energy, and the duration is measured over the time interval in which the energy grows from 5% to 90% of its total value.

Effect of Basin Depth and Distance from Basin Edge on Basin Wave Amplitudes and Durations

We have used the strong motion recordings of the 1999 Hector Mine earthquake recorded in the San Bernardino Basin to analyze the effect of basin depth and distance from basin edge on basin wave amplitudes and durations. As shown in Figure 8, the peak velocity increases markedly when the waves enter the San Bernardino basin, and grows in amplitude with increasing distance from the basin edge, even though the distance from the source is increasing. This is due to the trapping of body waves that enter the basin, generating surface waves. Once the waves have crossed the basin and left the basin, their amplitudes begin to decay again. A clear correlation of peak velocity with basin depth is shown in Figure 9. When depth increases away from the basin edge (i.e. when basin depth and basin edge distance are correlated), this can cause ground motion amplitudes to increase away from the basin edge.

The Arias intensity data (Figures 10 and 11) show trends that are generally similar to those for peak velocity that have just been described. The amplification in Arias intensity that is caused by the basin is significantly larger than for peak velocity. This reflects the increase in duration as well as in the amplitude of the waves that become trapped in the basin. The Arias intensity also has a stronger dependence on basin depth, with values increasing as the trapped waves propagate away from the source across the basin.

The duration of ground motion (Figures 12 and 13) also shows trends that are generally similar to those for peak velocity. The duration increases when the waves enter the basin, and decreases again upon leaving the basin but remains larger than the duration of the waves that entered the basin. Unlike peak ground motion, which tends to decrease with increasing distance from the earthquake, duration tends to increase with distance. The duration measurements on the basin edge parallel component have much more stable behavior than those of the basin edge normal component. This reflects the fact that the basin edge parallel waves, consisting of mainly of Love waves, have much larger amplitudes than the basin edge normal waves, which consist mainly of Rayleigh waves, due to the mechanism of the Hector Mine earthquake.

Development of Engineering Model of Basin Wave Amplitudes and Durations

Joyner (2000) developed a procedure for modifying standard spectral attenuation relations to account for the amplitude effects of surface waves in deep sedimentary basins. The objective of this project is to extend his work so that it is more broadly applicable in earthquake engineering. We are extending the model to include duration in addition to spectral amplitudes. We are extending the model to include basins other than the Los Angeles basin. In particular, we are including data from shallower basins, such as the San Bernardino, San Fernando, Santa Clara, and Eel River basins, in which the basin effects are expected to extend to shorter periods. We are extending the lower bound of the period range covered by Joyner from 3 seconds to 1 second, which will make the model relevant to a much larger number of structures. The result of this study will be a model, suitable for earthquake engineering application, that modifies standard ground motion models to account for the amplitude and duration effects of basin generated surface waves.

Conclusions

Polarization of Surface Waves

Basin waves are polarized predominantly in the directions parallel to and normal to the edge of the basin, with Love waves predominating on the parallel direction and Rayleigh waves predominating on the normal direction. This is consistent with the assumption made by Joyner (2000) and is caused by the lateral refraction of surface waves at the basin edge.

Differences between Basin Edge Parallel and Normal Components

There are significant differences between the amplitudes of the horizontal component parallel to the basin edge, which is predominantly Love waves, and the horizontal component perpendicular to the basin edge, which is predominantly Rayleigh waves, although these amplitudes will be affected by the relative strength of the incoming waves, which depends on focal mechanism and other factors. This is consistent with the assumption made by Joyner (2000).

Dependence of Amplitude on Basin Depth

There is a significant dependence of the ground motion amplitude on basin depth. When depth increases away from the basin edge (i.e. when basin depth and basin edge distance are correlated), this can cause ground motion amplitudes to increase away from the basin edge.

Duration measure

The Husid plot derived from the velocity waveform provides an appropriate duration measure that is independent of the absolute amplitude level.

Dependence of Duration on Basin Depth

There is a significant dependence of the ground motion duration on basin depth. When depth increases away from the basin edge (i.e. when basin depth and basin edge distance are correlated), this can enhance the tendency of ground motion duration to increase away from the basin edge.

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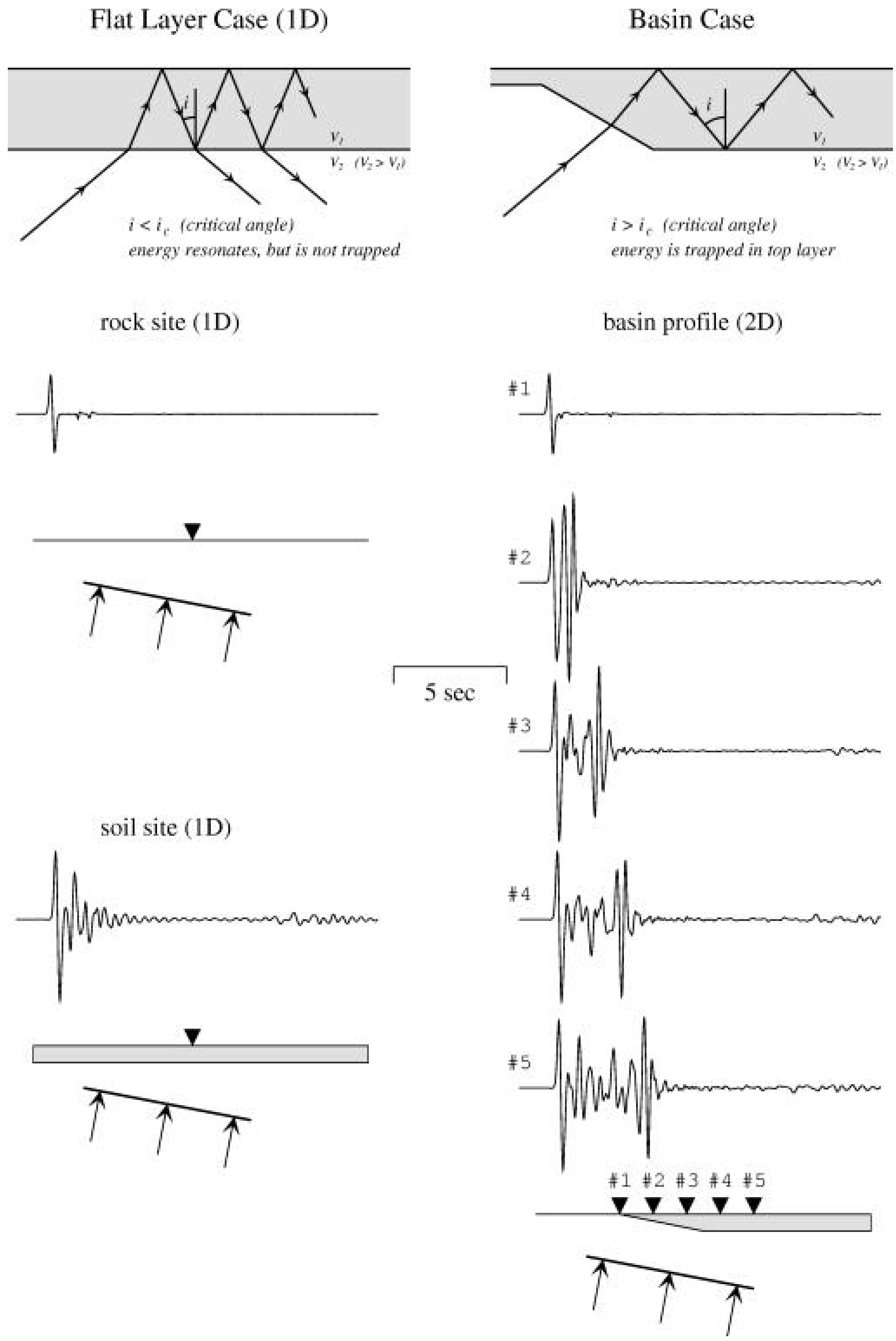

Figure 1. Schematic diagram showing that seismic waves entering a sedimentary layer from below can escape if the layer is flat (left), but can become trapped in the layer if it has varying thickness, for example when waves enter a basin through its edge (right). Source: Graves, 1993.

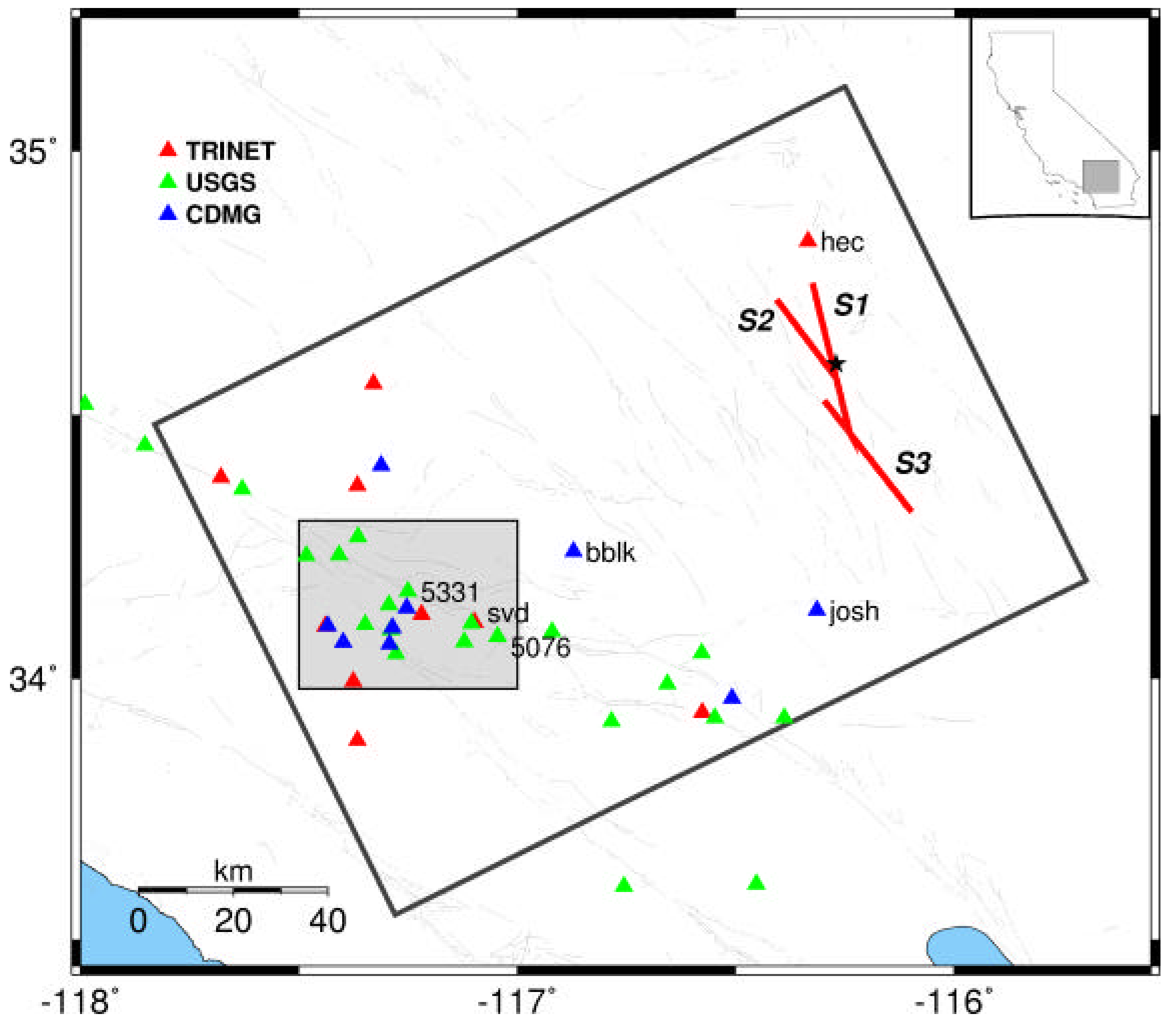

Figure 2. Map showing location fault segments S1, S2 and S3 of the 1999 Hector Mine earthquake, and strong motion recording stations including those in the San Bernardino Basin (small box).

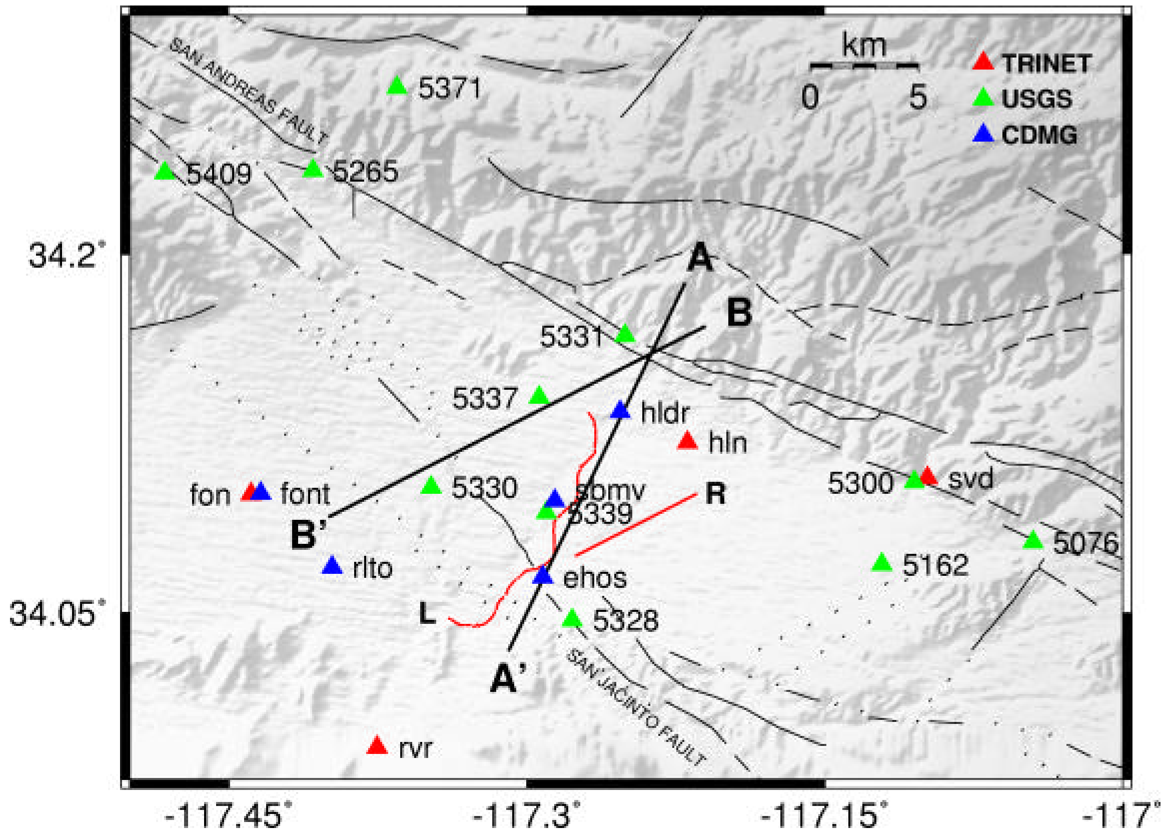

Figure 3. Locations of the San Andreas and San Jacinto faults, and strong motion recording stations in the San Bernardino Basin (small box in Figure 2).

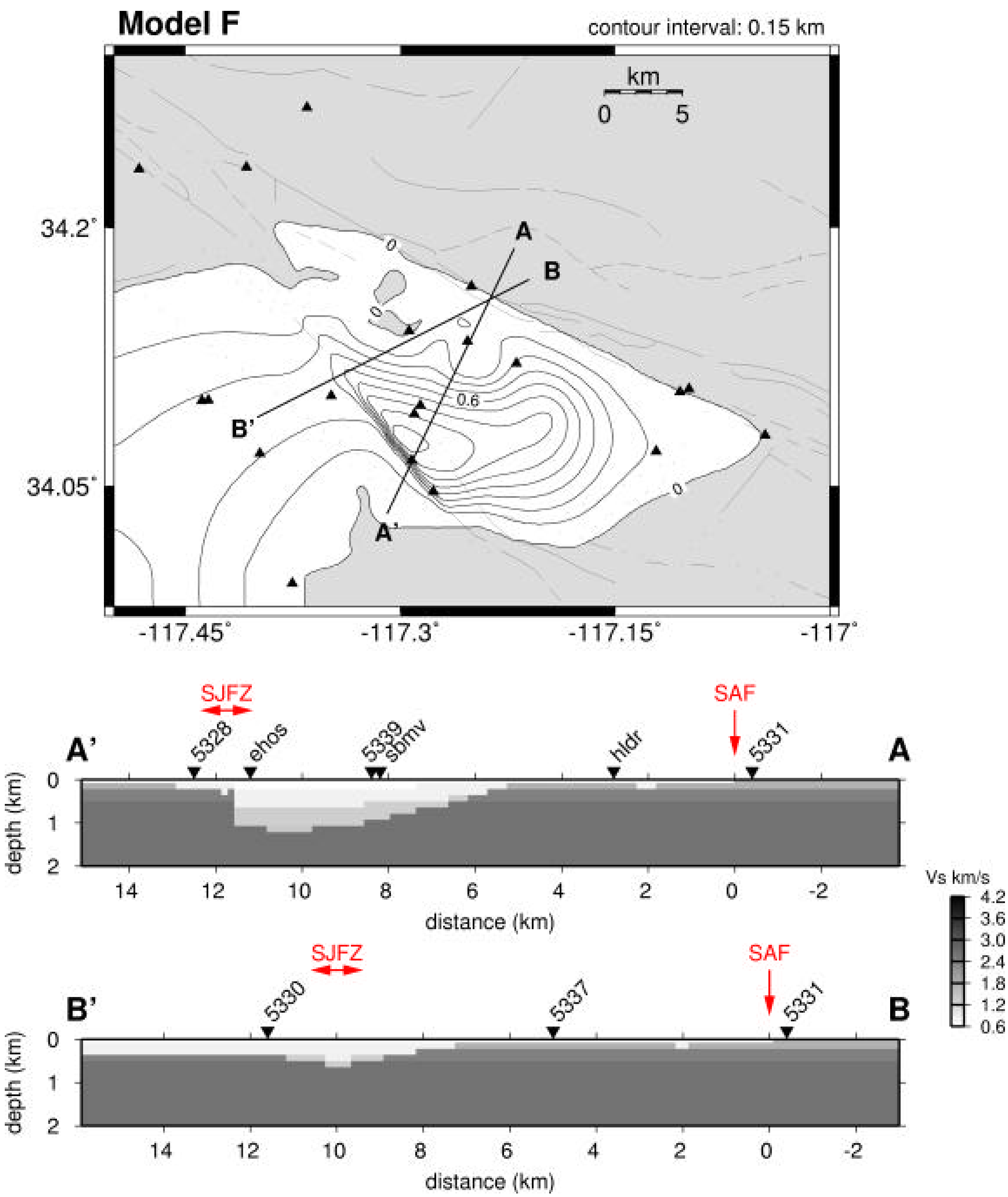

Figure 4. Map of the San Bernardino basin showing contours in depth to bedrock (top) and seismic velocity profiles along cross sections A-A' and B-B' (bottom).

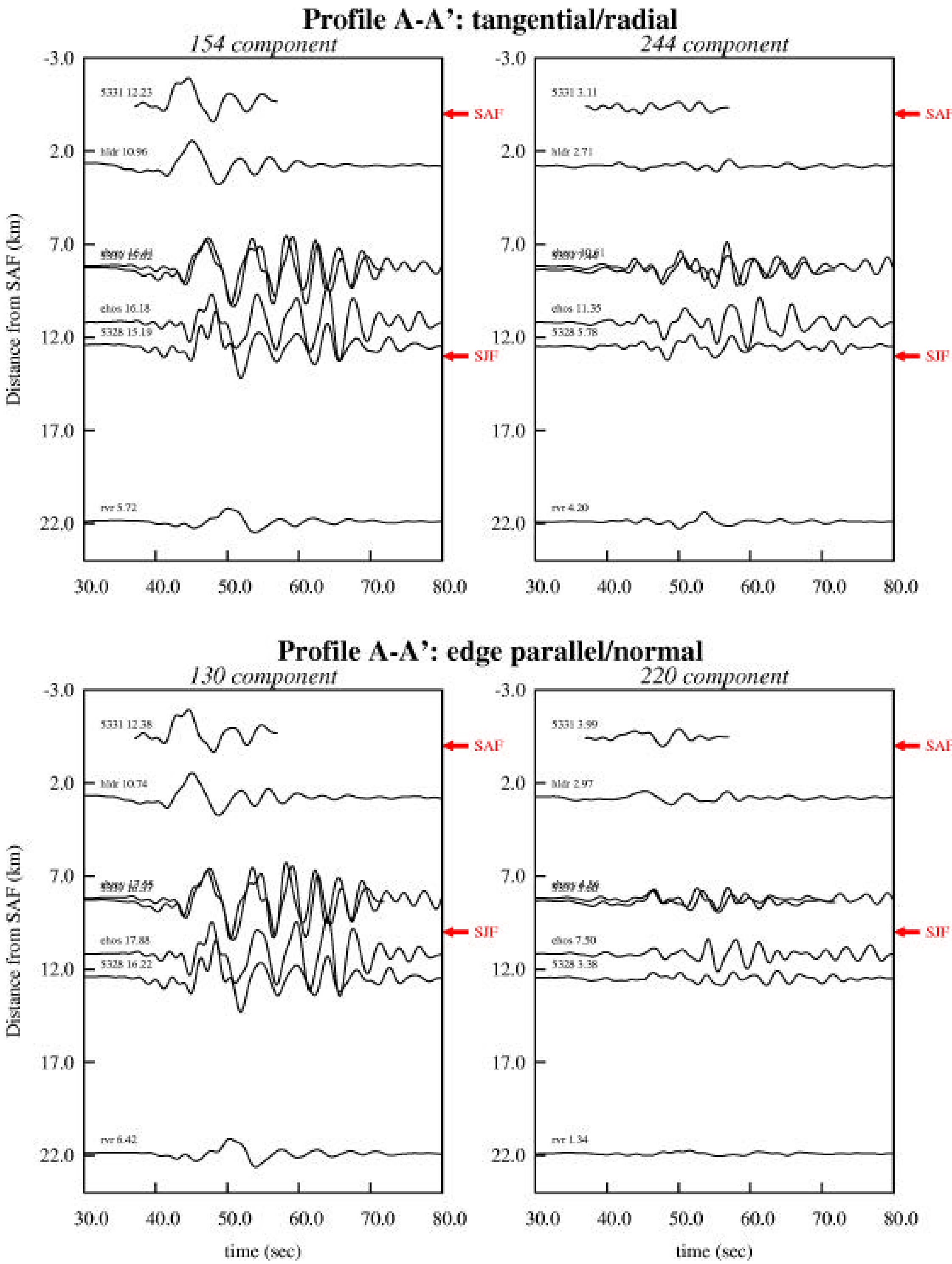

Figure 5. Top: Profiles of ground velocity of the 1999 Hector Mine earthquake recorded across profile A-A' in Figures 3 and 4 for the tangential (154) component (left) and the radial (244) component (right). Bottom: Profiles of ground velocity of the 1999 Hector Mine earthquake recorded across profile A-A' in Figures 3 and 4 for the basin edge parallel (130) component (left) and the basin edge normal (220) component (right).

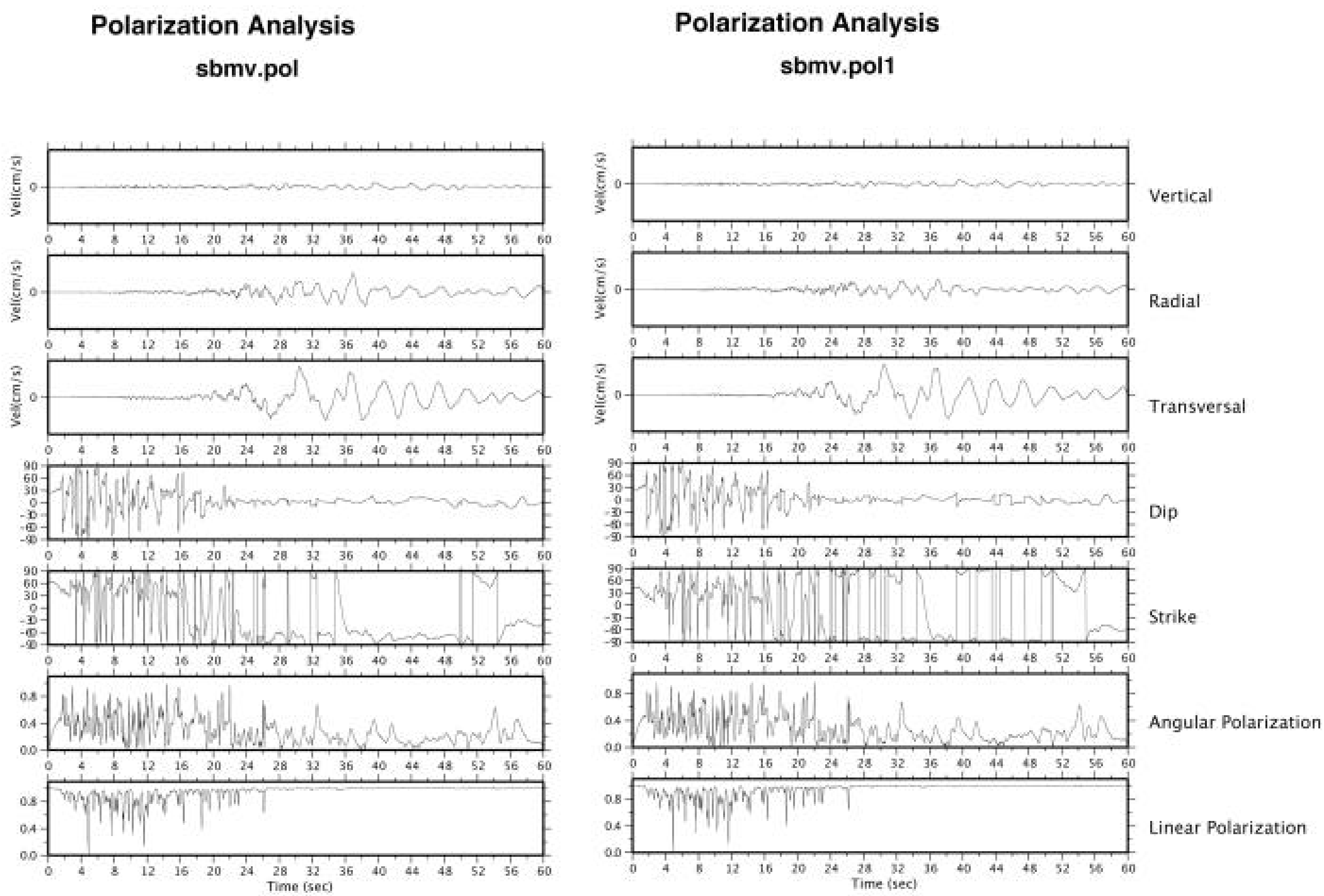

Figure 6. Polarization analysis of the Mountain View recording of the 1999 Hector Mine earthquake. The top three traces on the left are the vertical, radial and transverse components of velocity. The top three traces on the right are the vertical, basin edge normal, and basin edge parallel components of velocity. The bottom four traces on each side are the strike and dip of the maximum polarization, the angular polarization, and linear polarization of the motion, as defined by Vidale (1986). The strike for the basin edge normal and basin edge parallel components (right) is very close to +/- 90 degrees, but is about +/- 60 degrees for the radial and tangential components (left). This indicates that the ground motions are polarized in the basin edge normal and basin edge parallel directions.

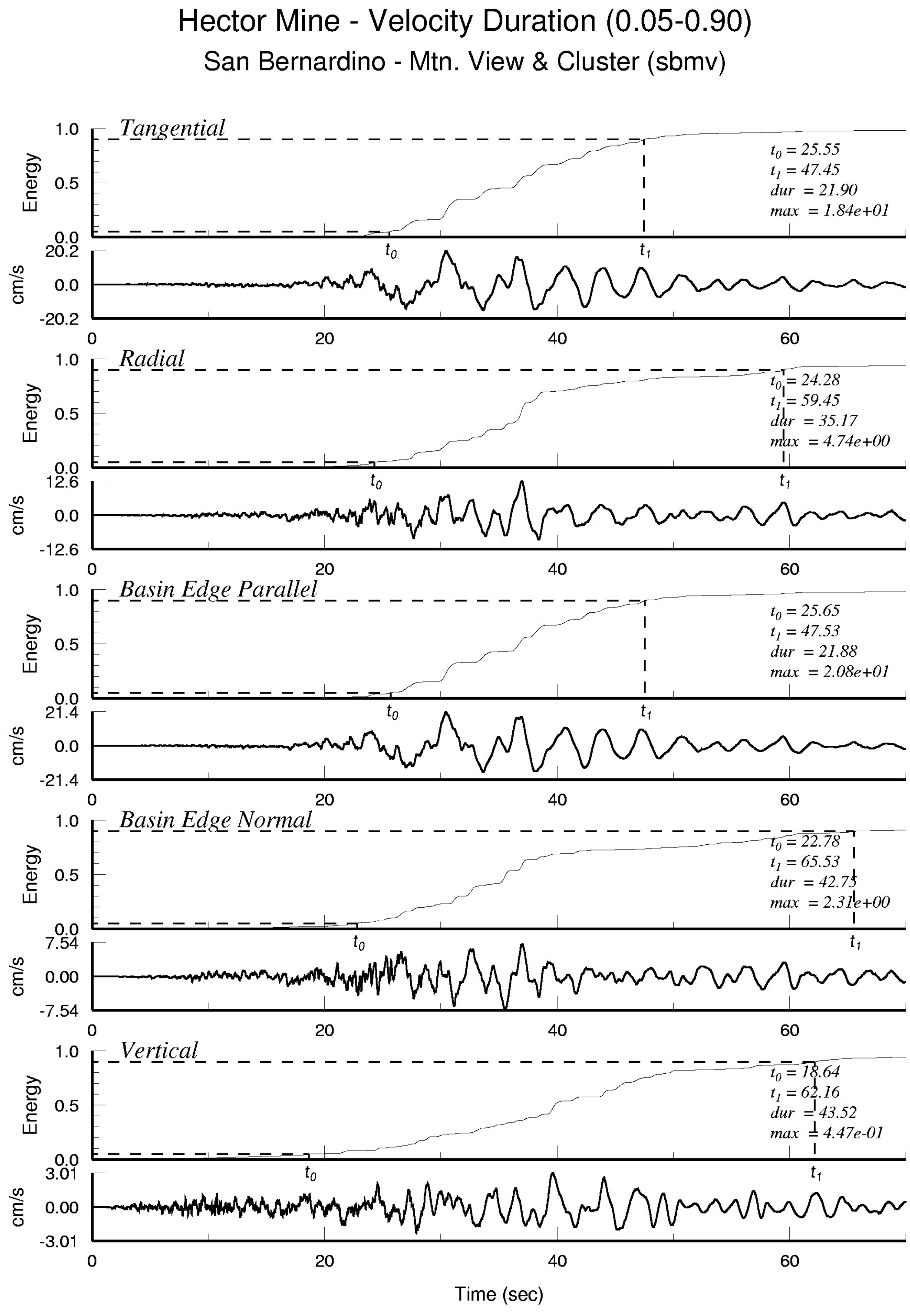

Figure 7. Husid plots of the Mountain View recording of the 1999 Hector Mine earthquake earthquake for five components of ground motion: tangential, radial, basin edge parallel, basin edge normal, and vertical. For each pair of traces, the top trace is the velocity time history, and the bottom trace is the Husid plot, showing measurements of duration based on the 5% - 90% interval of the cumulative energy.

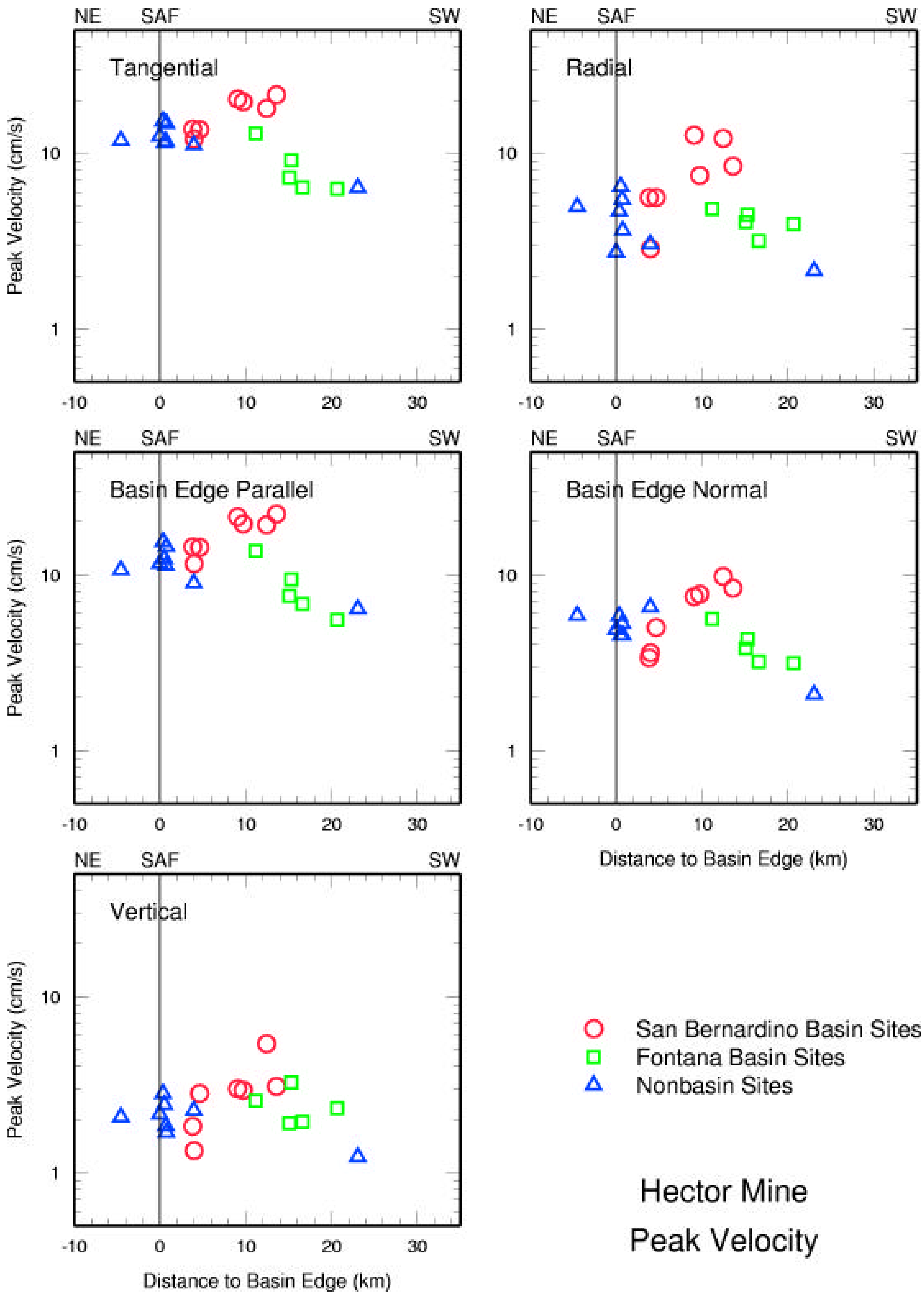

Figure 8. Distribution of peak velocity across profile A-A' in Figures 3 and 4 as a function of distance from the basin edge, marked by the San Andreas fault, shown by the vertical line, with positive values inside the basin, for five ground motion components.

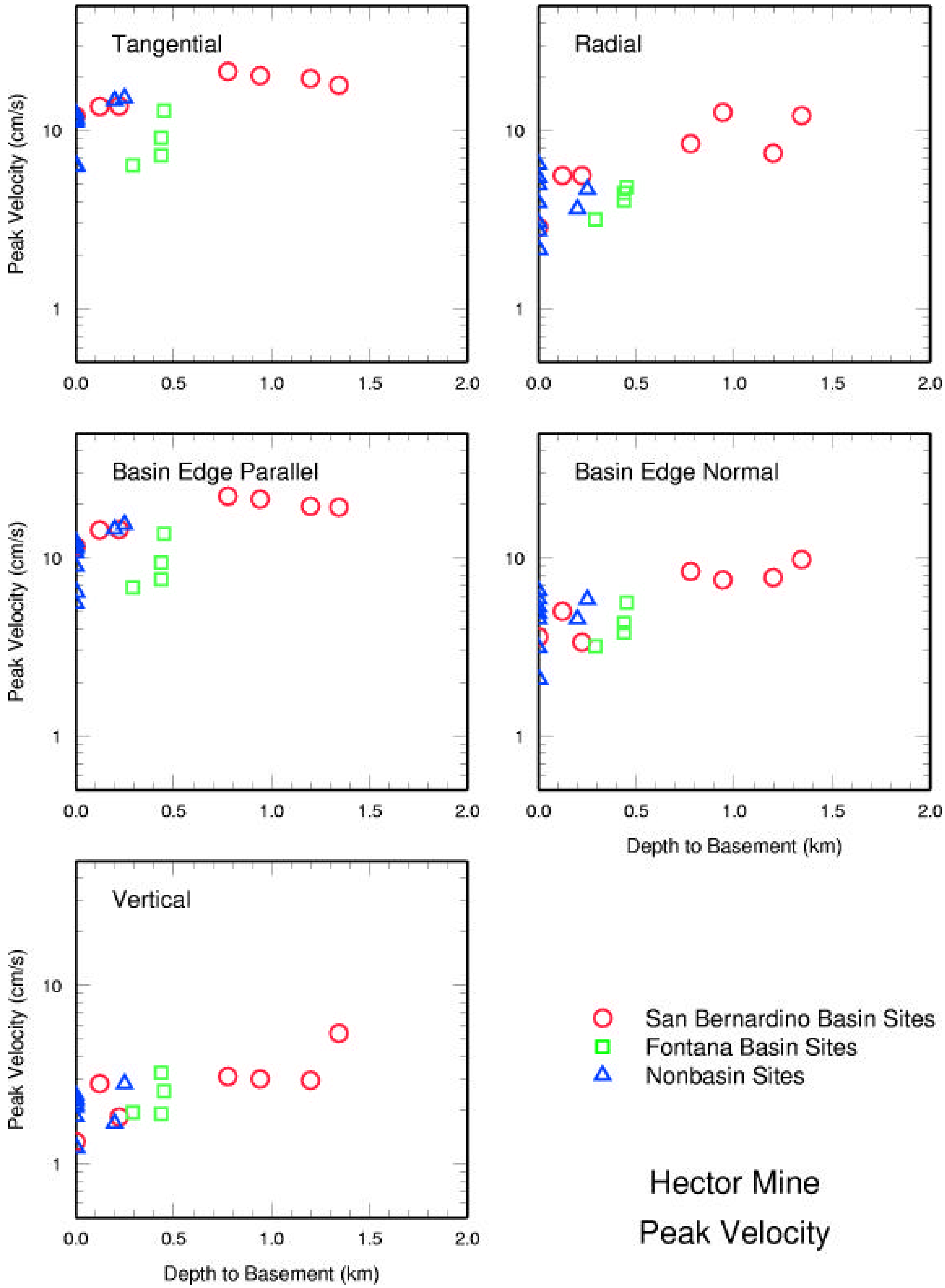

Figure 9. Distribution of peak velocity as a function of depth to basement for five ground motion components.

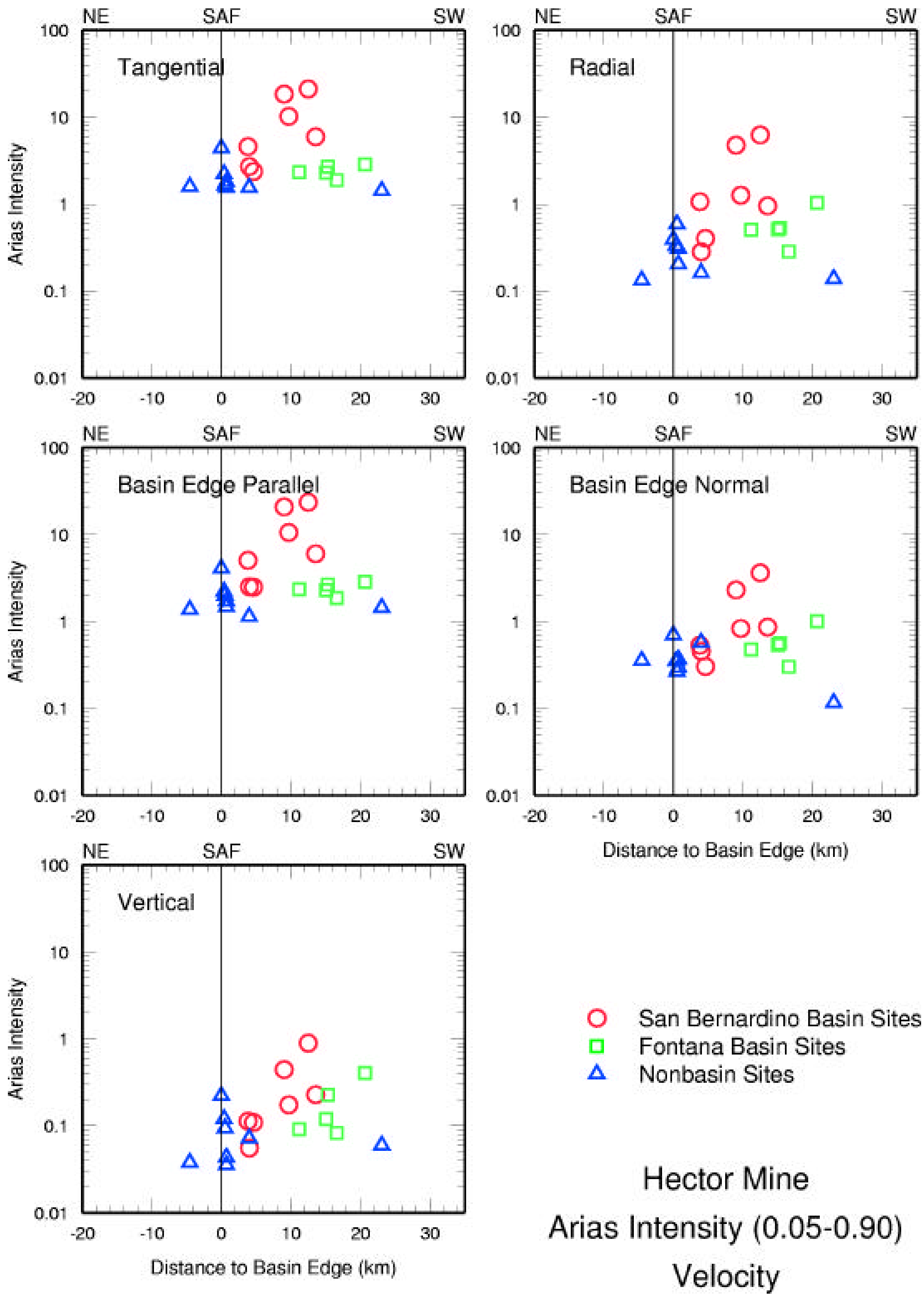

Figure 10. Distribution of Arias Intensity across profile A-A' in Figures 3 and 4 as a function of distance from the basin edge, marked by the San Andreas fault, shown by the vertical line, with positive values inside the basin, for five ground motion components.

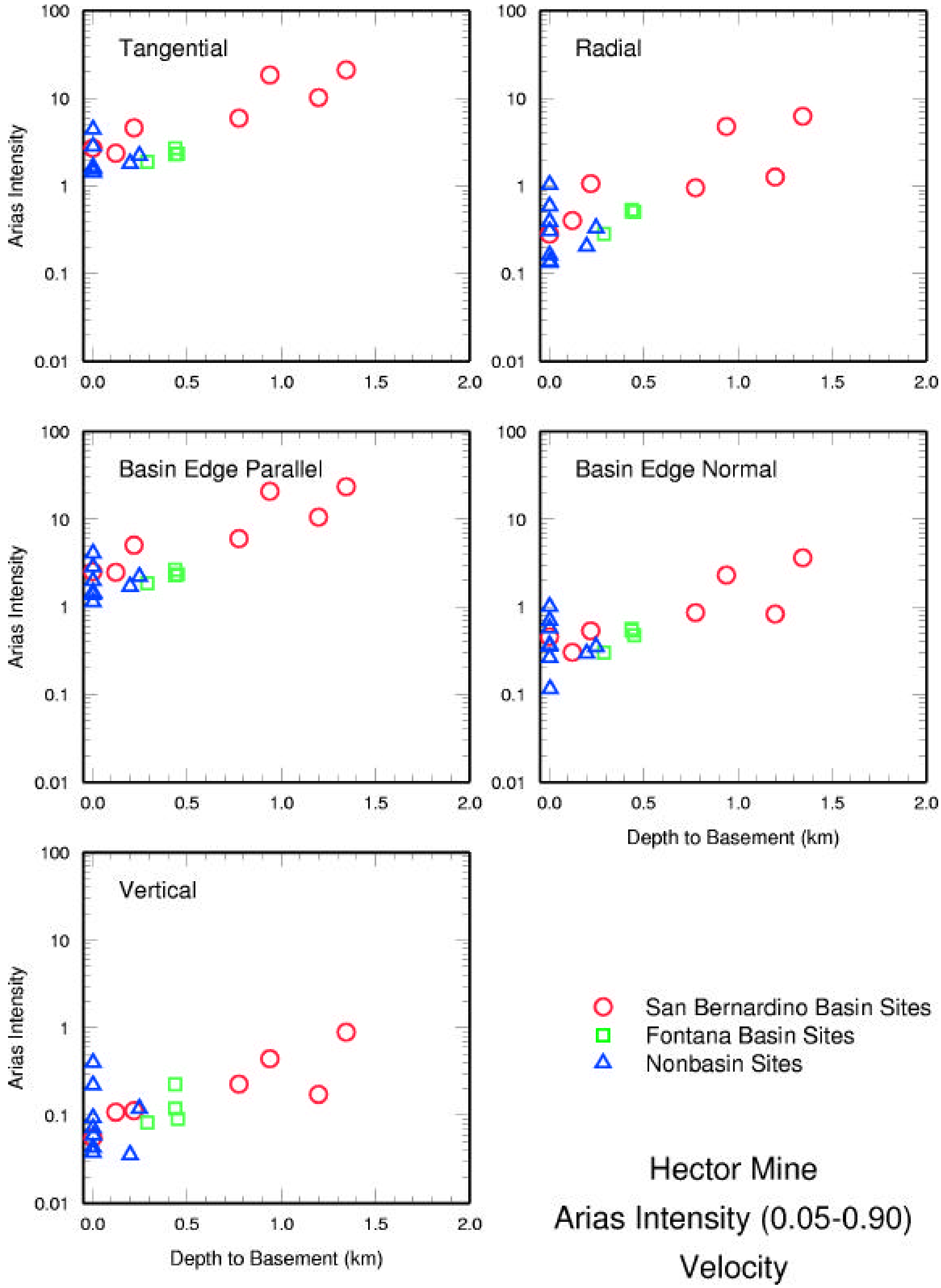

Figure 11. Distribution of Arias Intensity as a function of depth to basement for five ground motion components.

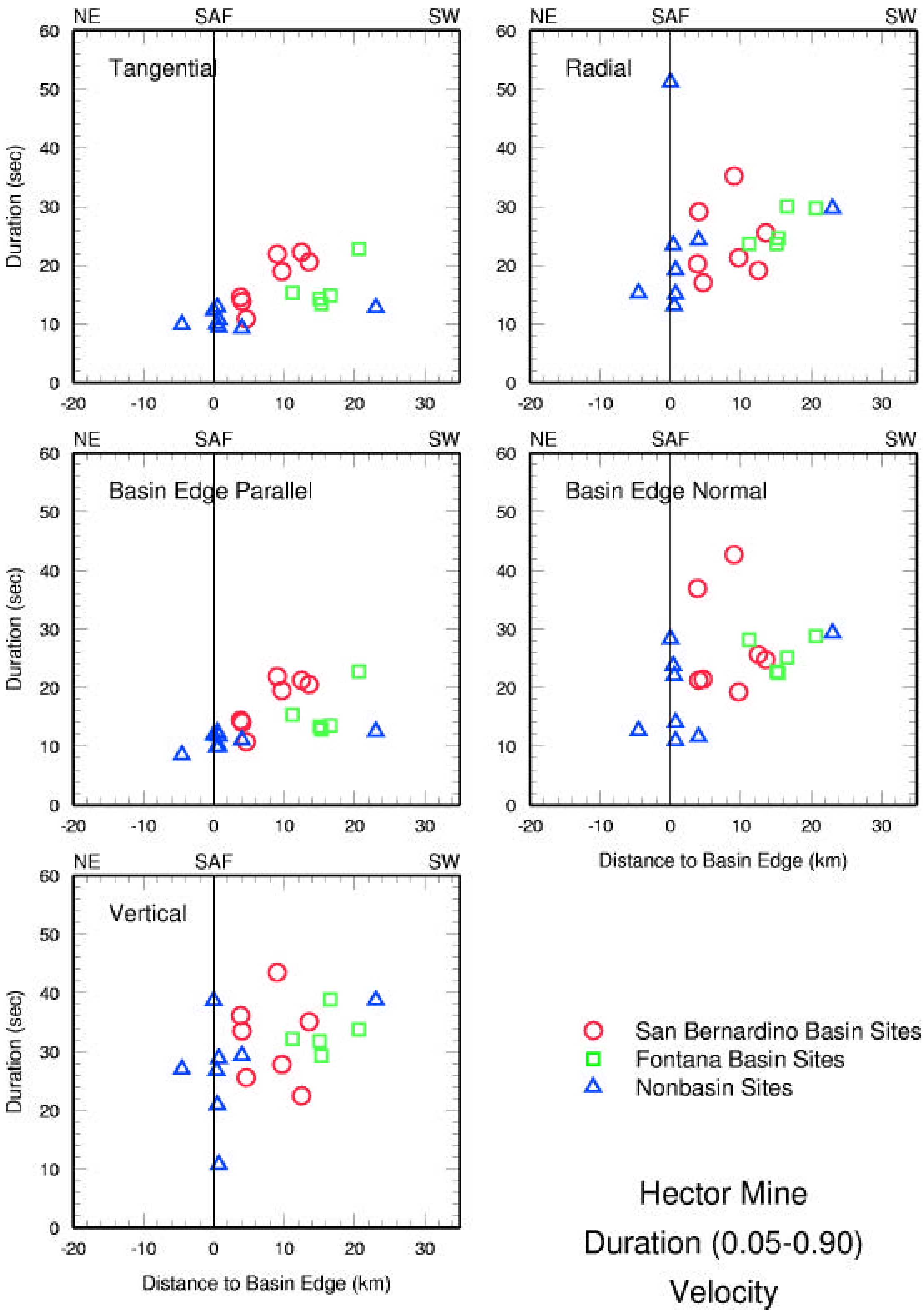

Figure 12. Distribution of Arias Intensity across profile A-A' in Figures 3 and 4 as a function of distance from the basin edge, marked by the San Andreas fault, shown by the vertical line, with positive values inside the basin, for five ground motion components.

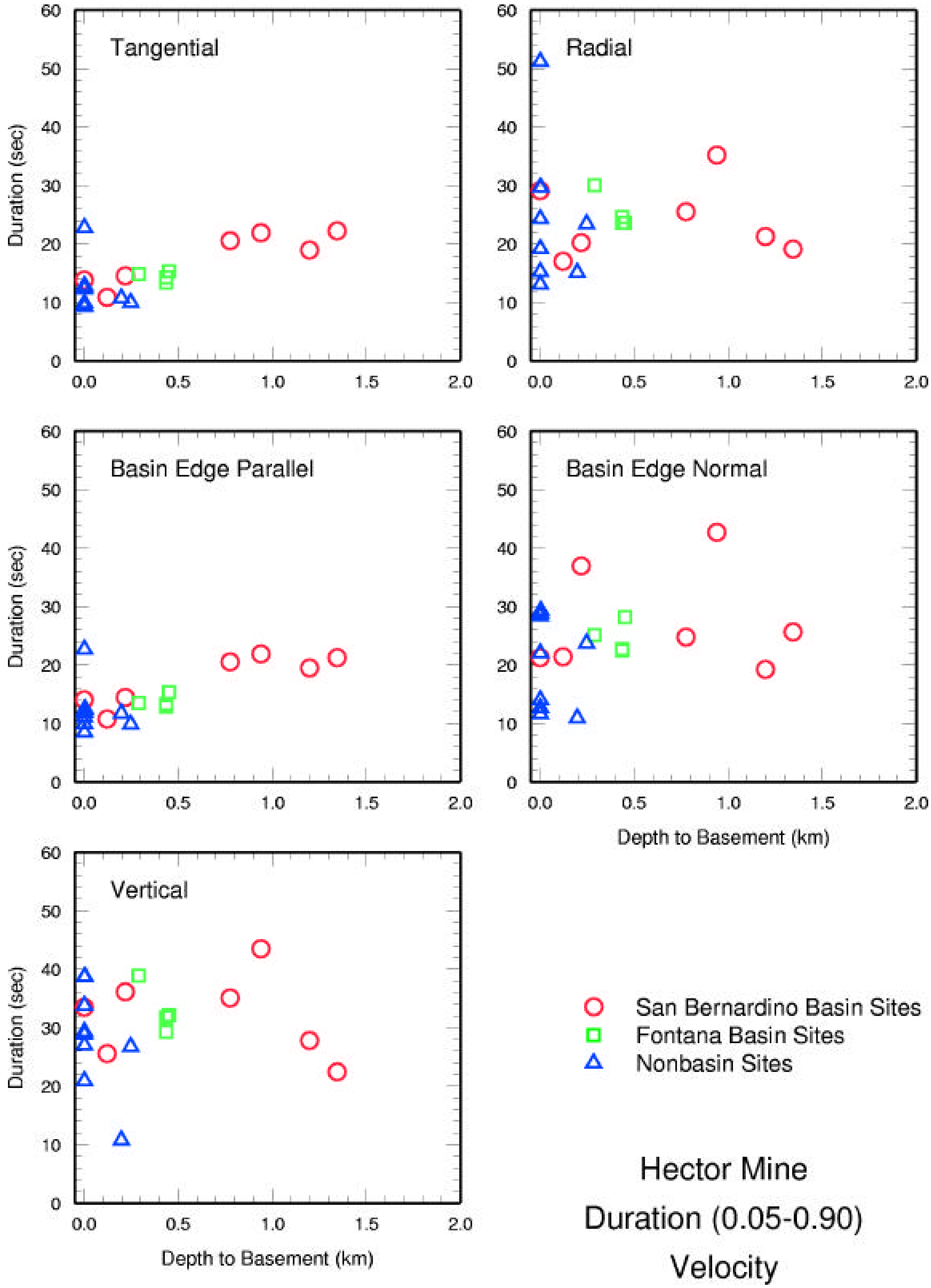

Figure 13. Distribution of Arias Intensity as a function of depth to basement for five ground motion components.

STRATEGIES AND CRITERIA FOR SELECTING BUILDINGS FOR ANSS STRONG-MOTION INSTRUMENTATION

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Abstract

This paper summarizes the conclusions and recommendations of the COSMOS workshop on Strong-Motion Instrumentation of Buildings held in Emeryville, CA, on November 14 and 15, 2001. The recommendations are intended to provide guidance for implementation of the strong-motion element of the Advanced National Seismic System (ANSS) and for strong-motion instrumentation of buildings in general. The guiding objective of building instrumentation should be to gain understanding of the behavior of representative building types in order to improve general predictive models, by: 1) challenging, verifying or calibrating models; 2) calibrating design and retrofit rules (codes); and 3) calibrating post earthquake evaluation rules. National and regional priorities and building selection criteria are needed. The ANSS should take advantage of advances in instrumentation and monitoring technologies and should build partnerships with state and local agencies and private sector companies in order to extend the scope of instrumentation of buildings.

Introduction

Adequate quantitative knowledge about the effects of earthquake ground motion on buildings can be gained only from instrumental measurements of motion both on the ground and in the buildings. The number of strong-motion recordings in buildings that have experienced significant damage in earthquakes is currently far too limited to permit development and validation of generalized predictive models. Currently, 207 buildings have reasonably extensive strong-motion instrumentation as part of the California Strong-Motion Instrumentation Program (CSMIP) and the U. S. Geological Survey National Strong-Motion Program (NSMP) (Huang and Shakal, 2002; Celebi, 2002). An additional approximately 500 buildings located in Los Angeles County have minimal instrumentation as specified by the Uniform Building Code guideline (Savage, 1997). And a few private sector companies have instrumented some of their buildings for the purpose of emergency response and recovery following a damaging earthquake (Otey, 1997). The total number of buildings that currently have adequate strong-motion measurement instrumentation amounts to less than three percent of the estimated 10,000 needed for all measurement purposes, including emergency response and recovery following a damaging earthquake (Stepp ed, 1997).

The Advanced National Seismic System (ANSS), which was authorized by the Congress in 1999, takes a step toward meeting the need for strong-motion response measurements in buildings. The legislation authorized 3000 three-channel instruments for placement in structures of all types (including buildings) and an additional 3000 three-channel instruments for measurement of strong ground motions in urban areas throughout the nation (USGS, 1998). The level of the ANSS authorization for strong-motion instrumentation of all structures falls far short of the estimated need for measurements in buildings alone. Nevertheless, the authorization presents a major opportunity to significantly advance strong-motion measurements. At the same time it presents a significant challenge for the community of earthquake professionals, requiring consideration of tradeoffs and development of implementation strategies that have the highest likelihood for improving earthquake engineering practice and advancing earthquake safety. Implementation strategies must 1) balance national and regional allocation of resources, 2) develop criteria and guidelines for selecting specific buildings in order to obtain recordings in representative building types and for specific measurement objectives, 3) stimulate advancements in instrumentation technologies, 4) identify actions to strengthen coordination between ANSS and building owners aimed at expanding private participation, 5) consider the needs for building health monitoring for emergency response and recovery, and 6) include plans and resources for data archiving and dissemination.

The workshop focused on strong-motion instrumentation of buildings as the priority need for the portion of ANSS resources allocated for strong-motion instrumentation of structures. Emphasis was placed on identification of knowledge gaps with respect to the types of buildings, the types of response measurements, and the national and regional allocation of resources that are together, likely to have the highest payback for advancing earthquake safety practice. Consideration also was given to opportunities for advancing monitoring technologies and to stimulating participation by building owners in strong instrumentation of their buildings. The workshop findings are intended to serve as the basis for development of strategies, criteria, and guidelines for optimally implementing the strong-motion element of the ANSS and for use in general for strong-motion instrumentation of buildings.

Summary of Findings

Building Types and Measurement Priorities

Predictive modeling is at the heart of building engineering. Predictive modeling is central to everything earthquake engineers do from post earthquake investigations to retrofitting buildings, to evaluating buildings, to designing buildings, to performance-based design.

The guiding objective of building instrumentation for response measurements should be to gain understanding of the behavior of representative building types in order to improve general predictive models, by:

- challenging, verifying, or calibrating models;
- calibrating design and retro-fit rules (codes); and
- calibrating post earthquake evaluation rules.

Building response recordings needed to meet this objective and advance earthquake engineering practice should determine measurement objectives for the population of buildings as well as for individual buildings, and should determine the number of instruments required in each building type, and the types of instruments to be installed. These together with consideration of the probability of obtaining recordings in a reasonable time (discussed below), form the basis for establishing criteria and developing guidelines for selecting buildings for strong-motion instrumentation.

The guidelines should require that specific measurement objectives be established for any and all buildings selected for strong-motion instrumentation in the context of filling information gaps to advance safety in earthquakes.

The most important measurement objectives are considered to fall into the following categories.

- Improving modeling of elastic response;
- Improving modeling of nonlinear response;
- Determination of inter-story drift;
- Determination of the torsional deformation;
- Determination of diaphragm deformation; and
- Determination of soil-structure interaction.

Criteria for instrumentation of specific buildings must address how many instruments are required to reasonably assure that a specific monitoring objective will be met. Workshop consensus was that 20 – 50 recording channels are needed, depending on the objective for instrumentation of a building, in order to obtain adequately complete building response measurements. For example, in order to be reasonably sure that sufficient recordings for the determination of inter-story drift will be obtained, placement of instruments on about one third of the building's floors is required. Assuming an average of 30 recording channels per building and three components for each instrument installation location, the currently authorized capital expenditure would permit instrumentation of 300 buildings. The currently authorized ANSS capital funding for 3000 three-component instruments for measuring response of structures, even assuming all of the instruments were placed in buildings, is clearly inadequate to meet the need.

Tradeoffs must be accepted for the development of guidelines and criteria for implementation of the system.

As an initial tradeoff it is recommended that instrumentation of buildings should be given high priority. Other structures – bridges, and other lifeline structures, dams and other critical facility structures – typically have independent requirements for strong-motion measurements. Other tradeoffs, discussed more fully later in this summary, include real-time monitoring of buildings for structural health assessment and for emergency response and recovery. The tradeoffs within the building inventory should take into consideration priority building types (considering current inventory, current construction and future trends), priority response measurement needs, and the likelihood of obtaining useful measurements in a reasonable time. It is recommended that the tradeoffs should emphasize instrumentation of fewer buildings with a scope of instrumentation that is adequate to reasonably ensure that established measurement objectives will be met.

The types of instruments selected for a particular building depend on the building type and on the established measurement objective. For example, displacement response measurements are needed for determination of inter-story drift. Strain measurements are required to determine the deformation of tilt-up wall connections. Other types of response measurements such as connection rotation or building torsion, require pressure gages and/or specific positioning of instruments within a building. Other objectives may require still other types of measurements, types of instruments, instrument configurations, or numbers of instruments.

Criteria for selecting specific buildings for instrumentation should be based on the considerations discussed in the preceding paragraphs. Specifically, selection criteria should be based on: 1) the number and value of the building types rather than on a simple sampling of the distribution of buildings of a given type in a region, 2) occupancy (office, hotel, hospital, and so on), 3) representative retrofitted building types, 4) foundation conditions, and 5) the potential for contributing to the objectives established for the ANSS building instrumentation program. Development of ANSS building selection guidelines should take the CSMIP (Shakal, and Huang, 2002) guidelines as a starting point. ANSS Regional Committees should contribute to development of the guidelines, and the guidelines should have a long time frame, national perspective.

All buildings selected for installation of strong-motion instruments should have a reference strong-motion station (COSMOS, 2001). The reference stations should be part of the additional 3000 urban strong-motion stations authorized as part of the ANSS capital expenditure for strong-motion instruments. This critical need emphasizes the importance of coordinated planning and selection of sites for urban strong-motion stations. The objective should be to optimize the location of strong-motion instruments installed for the purpose obtaining data for development of ShakeMaps that support response and recovery following damaging earthquakes, for example, with the need for building reference stations.

Resource allocation for instrumentation of buildings in different regions of the nation should consider the probability of obtaining recordings, but needs also to cover different building types, construction types, and any regional variations in code strength and ductility requirements. It is considered important that recordings of the responses of some of model building types—different construction types, different levels of strength and ductility – in lower seismic hazard regions also be obtained, if such model building types are sufficiently important and are not available in high seismic hazard areas. Because one model can span many building variations, however, the major consideration should be the probability of obtaining recordings that advance earthquake engineers' general understanding of the performance of building types that make up the building national inventory building.

Data collection, maintenance, archiving and dissemination must be considered key necessary elements of the building instrumentation program. The scope of data collection must include:

- metadata such as relevant information about the building and the site;
- building response recordings; and
- the damage state of the building associated with all recordings.

Real-time recovery of strong-motion recordings for the purpose of assessing building damage immediately following a large earthquake is an important safety objective and has potentially important economic payback. This important use of strong-motion recordings was recognized in the planning for the ANSS, which included consideration of instrumentation to support damage assessments for emergency response and recovery (USGS, 1998). Different points of view were expressed about the scope of instrumentation needed in a building to meet this important monitoring objective. There was agreement however, that to effectively meet this monitoring objective would require instrumentation of a significantly large number of buildings. The required number of strong-motion instruments would greatly exceed the current ANSS capital authorization for instrumentation of buildings. Nevertheless, real-time and near real-time recovery of building response recordings is considered to be a need that should be given continued attention in ANSS planning for the future. Consideration should be given to implementing building health monitoring as a revision of the Uniform Building Code guideline. Another option would be to give building owners incentives in the form of assurance of early resumption of building occupancy following a damaging earthquake. Importantly, continued attention should be given to developments in instrumentation and monitoring technologies in an effort to reduce the cost of strong-motion instrumentation of buildings.

Guidelines for Establishing National Priorities

Considerations for establishing national priorities should take account of the fact that the ANSS is a national program as well as the probability of acquiring useful measurements in a reasonable time frame. Considerations for selecting building types and for measurement priorities discussed in the preceding section generally apply throughout the nation, as any building response recordings obtained in one region will generally be transferable to other regions. This warrants giving higher weight to the probability of obtaining data in the development of an approach for allocating resources. The approach should also give appropriate consideration to the need for response measurements in specific regional model building types, different construction types, and different levels of strength and ductility in low seismic hazard regions. Recognizing that the ANSS is a national program, trade-off is required for a balanced national allocation of funding for instrumentation of buildings. As a long-term program goal, considering that the ANSS is a national program, it is recommended that 30% of the authorized 3000 strong-motion instruments allocated to structures in the ANSS Plan should be distributed equally among all the ANSS regions for instrumentation of buildings, with the exception of the Hawaii Region, discussed later. This allocation should be primarily for the purpose of obtaining response measurements of unique regional building types. The remaining 70% of the capital budget for instrumentation of buildings should be apportioned to ANSS regions based on relative regional seismic hazard/risk measures as discussed below.

Method I – Population Exposure to Seismic Hazard: For the purpose of estimating the amount of instrumentation required to reasonably ensure that adequate sets of measurements would be acquired in densely urbanized areas of the United States for *Hazard Mitigation*, Borcherdt et al., 1997, used population exposure to ground acceleration levels exceeding 0.1g defined by the national seismic hazard maps (Frankel, et al., 1996). The 1990 census was used for this analysis. Assuming that the geographic distribution of population is an approximation for the geographic distribution of the built environment, this approach was used to develop estimates of the number of instruments needed to ensure that the next major damaging earthquake is appropriately recorded. This approach was applied with regional network related weighting factors for purposes of developing the ANSS Plan (USGS, 1998).

The geographic distribution of population exposed to significant seismic hazard as an approximation for the distribution of the built environment, also provides a basis for specifying the geographic distribution of the approximately 300 buildings to be instrumented with the ANSS authorization. Assuming the number of buildings per cell of size 100 square kilometers is directly proportional to the percent of the total population exposed annually to peak acceleration > 0.1g yields the geographic distribution tabulated by state and ANSS region in Table 1. Without further adjustments of this distribution the highest priority region for well-instrumented buildings clearly is Region 1, California. Urban areas in Washington and Oregon, Utah, central United States and New York and neighboring states should be allocated fewer but nevertheless important numbers of instruments for installation in buildings.

***Method II – Annualized Earthquake Loss*:** Method II uses aggregate estimated annual dollar loss to the built inventory due to earthquake damage. FEMA (2001) developed an estimate of annualized earthquake loss (AEL) using HAZUS (1999, SR-1), which is a methodology for the computation of earthquake loss using up-to-date assessments of seismic hazard, building inventories, and building vulnerabilities. This geographic distribution of estimated earthquake losses to the existing built environment is considered more refined than that obtained by the population exposure measure alone.

High values of AEL are indicative of areas that have high seismic hazard and or high values of existing built inventory. The highest AEL is in California due to its high seismic hazard and large exposured built inventory. Other areas such as New York show significant AEL even though the seismic hazard is relatively much lower, because of a large exposed, high value built inventory.

In order to identify areas for which the built inventory, whatever its value, is exposed to high seismic hazard, the FEMA study computed an annualized earthquake loss ratio (AELR). AELR is defined as the AEL normalized by the total replacement value of the exposed inventory. Using the AELR measure gives more weight to hazard, resulting in areas with high seismic hazard, such as Alaska and Hawaii, having a higher index. A higher AELR index can be taken as a measure of a higher probability of obtaining building recordings in any time frame. By this index Alaska and Hawaii rank much higher, reflecting the fact that significantly higher proportions of the building inventories in these states are exposed to high seismic hazard.

The index for establishing the national distribution of instrumentation for building measurements should account for both the national distribution of AEL and the national distribution of AELR. The recommended index for allocating strong-motion instruments per state (assuming an average of 30 channels per building) is

$$Buildings_{30\ channels}/State = Maximum[AEL\%/State{:}0.50ALER\%](0.848)(300),$$

where 0.848 represents the percentage to normalize the index so that the total number of buildings is 300, based on 3000 3-channel instruments in buildings, which is equivalent to 30 channels installed in 300 buildings. The 50 percent applied to the AELR could be increased to place greater emphasis on those areas that have higher seismic hazard but not necessarily a large building inventory. The number of buildings implied by this index aggregated for each ANSS region in Table 1. Application of this index provides an objective method for national allocation of instrument resources. The results seem intuitively correct, based on a general knowledge of the national distribution of seismic hazard, the value of building inventory, and seismic risk. (Estimates of instrumentation for Puerto Rico could not be included, because assessments of seismic hazard had not been completed at the time of the studies by FEMA (Nishenko, 2001) and (Borcherdt et al., 1997). Recent completion of this assessment for Puerto Rico will permit estimates of AEL and guidelines for allocation of building instrumentation there in the future.)

The distribution of instrumented buildings implied by the above index is similar to that implied by the population exposure index, the principal difference being that the number of instrumented buildings implied for California is about 16 percent less than is implied by the population exposure index alone. Consistency between estimates derived using the population exposure index and the combined AEL and AELR index as two different methods provides additional assurance that the geographic distribution derived using either method provides a reasonable basis for assigning national priorities for the instrumentation of buildings. The distribution based on the AEL-AELR index is recommended as the most up-to-date basis for assignment of national priorities.

The consensus recommendation is that 30 percent of the capital expenditure for instrumentation of buildings should be distributed evenly among the ANSS regions in order to insure that regionally important building types can be instrumented. This implies that of the 300 buildings for strong-motion instrumentation, each ANSS region should have about 13 as a minimum. Reviewing the allocation based on the AEL-AELR index (Table 1) shows that the numbers of instruments allocated to each ANSS region meets this minimum number, except the Hawaii Region. Considering the moderate level of hazard and the relatively smaller exposed building inventory in the Hawaii Region, the full fixed percentage allocation is considered to be comparatively, not an appropriate allocation of resources. ANSS region allocations as indicated in Table 1 are recommended.

Guidelines for Establishing Regional Priorities

Guidelines for establishing priorities for allocation of resources within regions must accommodate the general guidance on building types and measurement priorities as well as such locally causative factors as earthquake magnitude, local foundation conditions, local variation of seismic hazard within a region, and local distribution of losses expected in an urban area at risk. The local and regional distributions of expected losses as calculated using HAZUS, provide a quantitative basis for assigning regional priorities and together with these additional considerations, provide the basis for development of criteria and guidelines for selecting specific buildings.

Workshop consensus was that HAZUS results should be used as the basis for establishing regional priorities based on the distribution of regional loss for maximum considered earthquakes in a region. The distribution of loss within regions for these events can be used to define areas and types of buildings most likely to be damaged. The percent of the total loss for each building type multiplied by the number of buildings allocated to the region based on the national priority appropriation provides a quantitative basis for allocation of resources for instrumentation of each building type. Borcherdt, et al., 1997, suggested a similar procedure based on ground motion estimates for a repeat of the San Francisco Earthquake of April 18, 1906 as a means of developing estimates for instrumentation of the built inventory in the San Francisco Bay area. Guidelines provided by this procedure should then be reviewed and interpreted by regional committees as guidance for selecting specific building types and locations and together with other locally causative factors, for establishing a ranking of priority installations.

Table 1. Priorities for national allocation of strong-motion instruments for buildings, by ANSS Region. The number of buildings is based on an average of 30 channels per building and the authorized 3000 3-channel instruments for structures (USGS, 1998).

ANSS Region/(States)	**Number of Buildings**
Region-1: California (California)	186
Region-2: Pacific Northwest (Washington, Oregon)	26
Region-3: Intermountain (Nevada, Utah, Arizona, New Mexico Montana, Idaho, Colorado, Wyoming)	33
Region-4: Mid America (Tennessee, Missouri, Illinois, Kentucky Arkansas, Indiana, Ohio, Mississippi Oklahoma, Texas, Louisiana, Michigan Kansas, Wisconsin, Nebraska, Iowa Minnesota, South Dakota)	13
Region-5: East & Northeast (New York, South Carolina, New Jersey Massachusetts, Georgia, Pennsylvania North Carolina, Connecticut, Virginia Alabama, New Hampshire, Maine Maryland, Vermont, Rhode Island West Virginia, Delaware, Florida District of Columbia)	22
Region-6: Alaska (Alaska)	14
Region-7: Hawaii (Hawaii)	7

Specific building types for consideration in addition to building types and measurement priorities discussed above, especially in high seismic hazard areas such as Anchorage are:

Steel Moment Resisting Frame (10-20 stories),
Steel Braced Frame (10-20 stories),
Reinforced CMU (5-14 stories),
Ductile Concrete Moment Resisting Frame (10-20 stories),
Concrete Shear Wall (10-15 stories),
Timber Building (Shear Wall) (5-stories), and
Special General Buildings (Large, complicated framing system).

Criteria and guidelines for selecting specific buildings should be integrated to the extent practicable with guidelines and criteria for selecting free field sites for strong-motion instrument installation in urban areas. The ANSS authorization allocates 50% of the capital expenditure for strong-motion instruments for installation in structures and 50% for installation in the free field for ground-motion monitoring. This allocation should be viewed as a long-term commitment for the ANSS rather than as a yearly requirement for allocation of resources by region and within regions. ANSS management and each ANSS region should have the flexibility to plan and prioritize resources considering the most urgent instrumentation needs identified by its Regional Committee as long as the 50% - 50% authorized target is met for the Program in the long term. Priority needs are expected to vary from region to region. In addition, each region should have the option to use some ANSS funds for mobile instrumentation for the purpose of conducting structural response studies as necessary. This portion of the instrumentation could be used, for example, to measure basic dynamic characteristics of given classes of buildings during ambient conditions or during small earthquakes that might occur frequently. Such data are lacking outside of California and are potentially important for determining whether certain code provisions, based largely on California data, are applicable to buildings in lower seismic hazard regions which may have different properties and may experience different ground motion characteristics.

In order to ensure that building measurement data needs are met, each ANSS region should appropriately constitute its Advisory Committee with at least a 50% earthquake (structural, and geotechnical) engineers.

Opportunities for Use of New Technologies

An important need is to optimize the costs of instrumentation at any point in time in consideration of the types of response measurements needed and by taking advantage of advances in instrument technologies. With regard to current technology needs for strong-motion accelerometers, discussions in the workshop developed the following recommendations:

1. +/-4g is sufficient for both Reference Station and in building accelerometers;
2. >4g should be considered for special measurements (e.g., impact/pounding, special buildings, equipment);

3. <4g may be sufficient for downhole measurements;
4. When high accelerations are anticipated, sample rate and frequency response must be appropriately increased;
5. 200 SPS is considered minimum for buildings in order to meet the need for higher mode information;
6. 16-bit resolution (sensors + recorder) is considered minimum for structural monitoring and higher resolution would be better.

Consideration of new technologies for building monitoring, as is the case for currently widely used technologies, requires a clear understanding of how the data are to be used in order to determine the appropriate building monitoring system technologies or monitoring configuration. An example already discussed is instrumentation for the purpose of obtaining real-time response measurements for assessment of the damage states of buildings following strong earthquake shaking. The public value of such measurements is sufficiently high to warrant continued investment in the development of effective monitoring technologies that are cost-effective enough to be attractive to individual building owners. In this regard real-time monitoring of some buildings or of some channels in selected buildings should be considered as part of the ANSS. For example, measurement of inter story drift is considered of primary importance. These measurements are not accurately obtained with current building instrumentation deployment. In the short-term, such measurements could be obtained by placing accelerometers or velocity sensors on adjacent floors. In the long-term, research is needed to develop new technologies for direct measurement if inter story drift.

Direct measurement of base rotation is considered to be of primary importance for understanding of soil-structure interaction. Instrumentation technologies for these measurements are currently available and should be installed in selected buildings.

In order to provide more flexibility as well as for reducing cost, it is considered desirable to replace cabled connection systems for instruments in buildings with wireless technology. Currently, however, wireless technologies do not have the distance range to completely replace cabled connections. Continued evolution of this technology is needed before wireless completely replaces cabling in a building monitoring system.

Other strong-motion monitoring technologies together with their potential importance for the short-term (current ANSS planning) and the long-term and are summarized in Table 2.

Table 2. Applicable Technologies for Building Monitoring

TECHNOLOGY	Short-Term Application	Long-Term Application
Lower Resolution Systems (Class B, 16-bit)	**X**	
Direct Rotation Sensors	**X** (low resolution)	**X** (high-resolution)
Soil Pressure Sensors	**X**	
Wireless Communication		**X**
Lower-Cost Accelerometers	**X**	
Strain Sensors	**X**	
Passive Peak Sensors	**X**	
Strong-Motion Velocity Sensors	**X**	
Static Tilt Sensors (SOS)		**X**
GPS Measurements	**X**	

Requirements for instrumentation of buildings for short-term assessments of damage states, coupled with adequately broad real-time data acquisition for purposes of supporting emergency response and recovery would require funding significantly larger than is currently authorized for strong-motion instrumentation. This important need should however, be given continued long-term attention. In particular, the possibility of developing more effective and lower cost instrumentation technologies should be given ongoing attention as part of the ANSS Program. This effort should in addition, target new technologies that can reduce the costs of instrumentation and monitoring while ensuring that the required data are obtained. Importantly, cooperative projects for instrument development should be developed with the NEES Program.

Considerations for Encouraging Private Participation

Private sector building owners assume that their buildings are properly designed and constructed and they are interested in strong-motion monitoring only for protecting their investment and maintaining operation. These essentially operational needs require considerations beyond the needs for building response monitoring. This building owner perspective needs to be engaged before private sector participation can happen. From this perspective, engaging private sector participation must be done in terms of needs and benefits as seen by the owners and operators of the buildings. The data that are collected must be demonstrated to be useful for processes associated with operational decision-making.

In terms of encouraging private sector organizations to get involved, there are some natural allies who can support this effort. First, there are owners who have instrumented their buildings or other structures and have operating experience using response results in decision-making and feel that they have had a successful experience. Second, there are experienced engineers who are advising building owners and have their own practical experiences with successful uses of building response measurements. These organizations/owners and practitioners are considered the best resources for any effort aimed at expanding private participation in strong-motion instrumentation of buildings.

Another important perspective that emerged is that there is a large gap to cross in order to educate private sector users of strong-motion data, as well as the providers of strong-motion data who are trying to meet their needs. Strong-motion monitoring has traditionally focused on providing building response measurements for dynamic modeling purposes. The products that are delivered by the providers respond specifically to the needs of this user group. In order to encourage building owners to invest in strong-motion monitoring, products that the owners can actually use must be provided.

An opportunity considered to have potential for expanding private sector participation involves demonstration projects, including co-funding or CRADA relationships, that get new organizations involved in building instrumentation projects.

Preparation of guidance for use of strong-motion data by building owners is needed and needs to be clearly communicated to the owners in order to bridge the education gap. ANSS should consider providing technical support for private owner participation in building monitoring by serving as recorder and distributor of data that would be collected. The building owner's needs for the data may be as limited as providing a free-field station's data that can go into ShakeMap. If instrumentation of a building meets the national and regional priorities of ANSS and the owner agrees to the use of the data for the public good, then the project would clearly contribute to ANSS goals. There is a range of possible building owner relationships with ANSS that would have to be developed on a case-by-case basis. Any building owner/operator who is engaged in any sense at all in getting and applying earthquake data becomes an advocate for the overall ANSS program.

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DEVELOPMENT OF ARCHIVING AND WEB DISSEMINATION OF GEOTECHNICAL DATA

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Abstract

This paper is a summary of findings and recommendations of a Consortium of Organizations for Strong-Motion Operating Systems (COSMOS)/ Pacific Earthquake Engineering Center (PEER) Lifelines Program workshop on archiving and web dissemination of geotechnical data held on October 4 and 5, 2001. The concept that emerged from the workshop is a central hub that would function as a virtual data center through which data providers share as well as disseminate their data. Presentations during the first day showed that the development of a virtual center for web dissemination of linked geotechnical databases can be largely accomplished using existing technologies to link the organizational elements of the system. The primary needs are to: 1) define the functional requirements of a virtual center, 2) define data formats, data dictionaries, indexes, and exchange standards, and 3) define and link the organizational components of the overall system. The principal consensus recommendation for future development was that, initially, a pilot implementation of a virtual data center should be developed. The pilot system should involve several of largest data providers including California Department of Transportation (Caltrans), Pacific Gas and Electric Company (PG&E), California Geological Survey (CGS), and the U. S. Geological Survey (USGS). Building on this pilot system, the links could be expanded to include other data providers and the general user community.

Introduction

Designing and constructing major parts of the built environment rely on expensive-to-collect geotechnical data on subsurface conditions (Bardet, 1998). Geotechnical investigations are routinely required for design and to obtain approval to construct significant buildings, highway bridges, dams and embankments, and other structures. Such investigations also support and are necessary for improving building codes and engineering practices, for developing and refining ground motion site response modeling, and for numerous research purposes. The data are generally collected following current professional practices. Consistent standards and quality practices are not generally followed, however. Much of the data collected have potentially general application and significant value for broad geotechnical engineering community and construction practice and for university research. The data typically reside in working files and archives of local, state, and federal agencies and private sector organizations. Limited efforts are being made by some data providers to archive data in searchable electronic databases. There are, however, significant barriers to broadly accessing these databases because they are held in multiple data formats, data archiving and dissemination methods are incompatible, and the cost of retrieving data from paper records is high.

Newly available and emerging computer and communications technologies are making possible economical storage and sharing of valuable data to better serve society's needs. COSMOS and the PEER Lifelines Program held a workshop on October 4 and 5, 2001 to document current archiving and web dissemination technologies and to define future developmental needs in order to archive web dissemination of linked geotechnical databases. The workshop was motivated by the recognized economic importance of having geotechnical data readily accessible to the broad community of users. The objectives of the workshop were to develop consensus recommendations for classifying, archiving, and web dissemination of various types of geotechnical data and to develop a plan of action leading to development of a web-based virtual database system linked to multiple databases. Discussion papers together with the complete conclusions and recommendations of the workshop are published in a proceeding (Stepp, et, al., editors, 2002).

A generalized overall concept of how a web-based virtual data dissemination center could be set up is illustrated in Figure 1. The virtual data center hub would not house the data, but could house metadata and/or data indexes and translators that allow data to be accessed through the hub from various linked databases. The concept is that the data sources or providers would also be users and the general user community could access data from all databases through the system hub. Elements of a virtual center are elaborated and development needs are discussed in the remaining sections of this paper.

Archiving and Web Dissemination Architectures

Web Dissemination Architectures

Perhaps the most important finding of the workshop is that architectures for web dissemination of data from multiple databases are readily available. Developments needed for their implementation primarily involve the details of linking databases and accessing data. Two examples of applicable data dissemination architectural schemes identified as being most promising are "federation" and "harvesting". Diagrams displaying the basic elements of federation and harvesting architectures are shown in Figures 2 and 3 (Futrelle, 2002). In the federation architecture, the databases (data providers) themselves provide the search services directly to the end users (Figure 2). In the harvesting architecture, the query capabilities are maintained at the central hub of the system (Figure 3).

In the federation architecture each of the databases must develop and maintain the capability to execute user queries (Figure 2). The user performs the search by searching each of the databases in the system and the queries are done "live". This requires a standard query interface, a standard query language, a standard query protocol, and a standard set of fields to search that must be common for all participating databases. Each linked database must have its own index so that it can perform searches and queries in a reasonable amount of time. Consequently, there must be substantial agreement among the linked databases in terms of what query services are provided. This architecture is advantageous if the data are time critical. Also, there is no duplication of metadata within an index and no need for a central hub to maintain copies of metadata. In emergency response applications, for instance, federation might be the best architecture.

The harvesting architecture, depicted in Figure 3, provides a uniform user interface for querying multiple databases using common indexes, and for obtaining uniform communication of data. In order to implement this architecture the data providers must export or link the metadata from their databases into the harvester hub, which links users to multiple databases. The method to accomplish this must be designed and developed. Figure 4 illustrates in schematic one way a harvesting architecture could be designed to implement a web-based virtual geotechnical database system. In this example, there are four data providers (i.e. data sets), each generating metadata from their own original data sources. The generation and maintenance of the metadata must be the responsibility of the data providers. A defined set of fields or parameters would be exported to the index in order for data to be queried by the harvester. (A data index is a system such as an RDBMS or perhaps versioning software, containing maintained linked lists of the data and metadata that may be retrieved from the databases using the harvester hub.) The data providers must at least have those fields, which must be defined in a common standard for all databases, in their data dictionary. There should be no restriction on fields or parameters that can be exported to the index to be queried by the harvester hub. That is, the harvester hub should allow a user to search on all of the fields in the individual databases, as well as the common overlap described by the standard format. Extensibility can thus be built into this system architecture.

Figure 5 is an expanded schematic showing the elements of a web-based virtual geotechnical database system based on the harvesting architecture. Elements consist of multiple distributed databases maintained by data providers, metadata, data indexes, data translators, and a harvesting hub, which interfaces with users. Metadata are defined as data that describe data, encode relevant semantics, and are optimized for exchange (Futrelle, 2002). Retrieval is actually done at the indexes, which provide random access. Thus the harvester keeps track of the metadata from all of the different data sources. There is one subset or intersection that the harvester knows about and uses to link the user queries to the data. A data index works with the end users' queries by pointing to the data of interest. Though indexes could be maintained either by the data providers or the harvester hub, the data providers could best maintain their indexes, since they understand their data and how the data are organized. Each data provider's data dictionary and thus metadata should conform to a translator. The data translators, which could be maintained either at the harvester hub or by the data providers, filter the retrieved data or metadata into a standard format for dissemination through the virtual center hub to users. Dashed lines and dotted lines indicating the two possible system designs illustrate these alternatives. Different indexes can be developed to harvest different information. The end users should be able to access indexes as well as any sub-indexes within the virtual system that satisfy the requirements of their applications. Other applications besides metadata generation can be attached to a given database. Software already exists that can be used to set up a harvesting scheme such as Figure 5. This is only one scenario; there are other possible variations in terms of the number of data providers and indexes.

The harvesting architecture depicted in Figure 3 and expanded in Figures 4 and 5, is considered to have significant advantages for linking distributed databases through a central hub with a common user interface. For this reason this architecture is recommended for development and implementation of a web-based virtual geotechnical database system.

User Scenarios

The definition of user scenarios was identified in the workshop discussions as a priority need in order to establish functional requirements for a virtual database system. User scenarios identify patterns of geotechnical data use and users. User scenario-based design of the virtual system establishes what virtual system architecture will be required, what kinds of services and software need to be developed, and possibly other needs. The broadest range of user scenarios should be developed in order to establish the basic needs for the design of the virtual system. Focus should be on scenarios that critically depend on integrated data use across several different domains or sub-domains, data formats or instrumentation, and geographic or institutional locations, and should cover most data uses. It is critical that data domain specialists representing the principal data suppliers participate in developing user scenarios, and information technology (IT) specialists should participate in the design of the virtual system.

Data Dictionary and Formatting Standard

Explicit user scenarios also form the basis for development of a data dictionary and formatting standard, a critical element of the virtual system. The development of a strawman data dictionary from existing dictionaries would be a good way to begin this process while concentrating on content, or parameters of interest. Exiting dictionaries include the AGS (Association of Geotechnical and Geoenvironmental Specialists, UK) NGES (National Geotechnical Experimental Sites, UNH) geotechnical data standards, and LAS (Log ASCII Standard, CWLS). The AGS standard represents the practical, applied field side of geotechnical engineering, while the NGES standard represents the more academic or research side. A strawman data dictionary was developed from existing dictionaries and presented as a structured parameter list for this workshop (Turner and Roblee, 2002).

The recommended approach for developing a data dictionary and formatting standard that emerged from the workshop is schematically depicted in Figure 6. The approach is illustrated by example with a virtual system having two databases, database A and database B. (Ultimately the envisioned system would include a number of databases.) The need is to define the common overlap in data dictionaries or parameters of interest. This would require agreement on a common definition of the parameters in the standard. The overlap is the information that would actually be transferred and exchanged. Care should be taken not to try to encompass all parameters included in every existing standard studied, as the resulting standard would become unmanageable. Although the need is to create a standard data format, the data exchange system or format itself should be flexible enough so that end users are able to use the data however they want. The main purpose is to make the data easy to exchange and in a format that participants actually want it for specific uses. Thus both content and semantics need to be defined to produce a usable format.

Data Exchange Standards

Data exchange standards (metadata, data indexes, and data translators) are other major components of a virtual database system that must be given priority development. Data exchange is accomplished by use of translators, which are filters between the data at a database participating in the virtual system and data retrieved through the virtual geotechnical system hub. In physical terms, translators are basically software applications. The definition of the translators will depend on:

1) The requirements for the data producers to generate metadata and convert their data to the chosen standard format for exchange; and
2) The overall information architectural scheme chosen for the virtual system.

Another important need is to establish where the translators and indexes will be located and maintained. These tools could be part of the providers' database systems or of the virtual system hub. These alternatives are further illustrated in Figure 5. The dashed lines and dotted lines indicate the two possible system designs in terms of responsibility. In the dashed line design, the virtual system hub is responsible for maintenance of the data provider's indexes, metadata and translators. Whereas in the dotted line design, the data providers are responsible for their own indexes, metadata, and translator applications. The most appropriate design is for the data providers each to maintain their own translators and indexes with guidance from the virtual system hub. The virtual system hub could develop the indexes and translators, then turn them over to the data providers. In the alternative design the data would end up being centrally managed, which is more problematic. In order to include participation of smaller agencies or professional firms who may want to participate, a combination design would be appropriate where the harvester points to data from the bigger agencies and also stores data from the smaller agencies or companies centrally.

It is important to note that translation of original data into metadata and data in a standard format inevitably involves data loss. A combined format and index will normally contain less information than all of the individual databases that were integrated. Otherwise the functions of those databases would have to be restricted, which is not desirable since the databases could also have other applications attached to them.

An important, though less critical, issue is the format for communicating the data to the end users. This could be via XML, ASCII or some other format. A distinction must be made between the syntax of the data standard (the structure of a line string in some language), and the semantics (the meaning of a line string in some language) of the data standard. The semantics are what each element in that data standard means. As an exchange format ASCII is the most commonly used, and is employed in the AGS, NGES and LAS formats. The recommendation of the workshop is to use ASCII as the exchange format, since it is universally accepted, and to use XML for metadata. Notwithstanding, all format options should be investigated as part of the implementation process. As for communication of the data, for instance by developing viewer applications to display on-the-fly graphics, internet formats such as XML, XSL, and VML could be used, or even image formats such as .gif files could simply be generated by invoking an existing software tool so end users could view data from the central hub. The development and maintenance of such viewing applications would be the responsibility of the hub.

Implementation Actions

This section describes a plan of actions needed to implement a virtual geotechnical data center. The actions focus on development of applications for already existing technologies. That is, the archiving and web dissemination system should use existing database technologies and web dissemination architectures and develop applications needed to implement these in a web-based dissemination system. The harvesting architecture emerged as the most promising for implementing a web-based virtual system. The elements of a web-based virtual database system based on the harvesting architecture are depicted in progressive detail in Figures 3, 4, and 5. Based upon the harvesting architecture, the implementation actions address elements of the system including, 1) definition of the functional requirements of the system, 2) development of a data dictionary and data formatting standard, 3) development of data exchange requirements and standards (handling of metadata, data indexes and data translators). As an implementation strategy initially, a pilot system should be developed linking the databases of two or three data providers. Once the pilot system is operational, implementation could be expanded to include the broadest participation.

Definition of the functional requirements of a web-based system

The central concept and functional objective is a web-based data system that facilitates data dissemination from participating distributed data sets, functioning together as a virtual data system. Definition of the functional requirements for the virtual system is a priority primary need. The important first step is the identification of the data users and data user scenarios, which in turn determine details of the functional requirements of the system such as the scope of the standard data dictionary, the method of handling metadata, and the scope and method for handling data indexes. The identification of data users and data use scenarios must involve the participation of data providers. Geotechnical data providers normally also maintain databases which may be linked to a web-based dissemination hub. Once the functional requirements of the system have been established, the most appropriate implementation of the harvesting architecture can be determined.

A work group constituted of data providers and users can best define the functional requirements of the system. The work group should identify data users and data use scenarios, and use this information to establish the functional requirements of the web-based system, such as depicted schematically in Figures 3, 4 and 5. The work group should include an IT specialist who has experience with the harvesting architecture. As a starting point for developing a comprehensive set of user scenarios, a catalog of the types of available existing data and where the data can be accessed should be developed.

Development of a Data Dictionary Standard

A data dictionary standard is a fundamental need for implementation of any database, and a fundamental requirement for linking distributed data sets in a web-based virtual system. Data providers develop diverse types of data including geological, geophysical, geotechnical, and geoenvironmental. The data are collected for a multitude of purposes ranging from geotechnical characterization of specific sites, to hazard mapping, to general geological mapping, to specific research projects. In the absence of any standard, data are held in diverse formats and critical information that is needed to inform the general user about the data and permit any evaluation of its quality is most often not documented (Bardet, 1998).

Issues relative to developing a standard data dictionary vary depending on whether the standard is for the electronic capture of legacy data held in paper form, data in an existing electronic database, or data to be collected in future projects. Legacy data can be captured in electronic format following a data dictionary standard, but significant metadata needed to evaluate its quality normally will be missing and cannot be recovered. Existing electronic databases have their own data dictionaries. These dictionaries may intersect as depicted in Figure 6, permitting a limited range of common access through a central hub. On the other hand, a common data dictionary standard covering the full range of data and data use scenarios could be developed and used by all data providers for data capture in the future. Such a standard could facilitate the direct capture of field data in electronic format and thereby insure that the appropriate metadata are collected at the same time.

For the purpose of initial implementation a critical need is to develop a data dictionary standard that includes the largest intersection of the dictionaries of existing electronic databases and has built in flexibility for expansion. The standard could then be used for purposes of capturing legacy data in electronic format and as the basis for implementing a web-based virtual system for geotechnical data dissemination linking existing geotechnical databases. Two data dictionary standards are available: the AGS Standard and the LAS Standard. The NGES Geotechnical Database used the AGS Standard as a starting point for the purpose of developing the NGES data dictionary (Satyanarayana and Benoit, 2002), and a strawman data dictionary adapted from existing data dictionaries has been developed by Turner and Roblee (2002).

The NGES data dictionary should be adopted as the starting point for development of a data dictionary that incorporates as broadly as achievable the intersection of existing data dictionaries. The strawman data dictionary developed by Turner and Roblee (2002) could be considered as an alternative starting point for this action. The data dictionary should be flexible to permit expansion as the virtual system is linked to additional databases.

Because of the broad range of types of geotechnical data, the task of developing a data dictionary must necessarily involve a broad range of data specialists. Issues involved with geologic descriptive data, boring log data, and laboratory test data, to cite a few examples, differ significantly and the collection of these data involves specialists with different backgrounds and experience. In order to address these issues in the development of a data dictionary standard, an appropriately wide range of data specialists must be engaged. These should include for example, specialists in geologic non-parametric data, *in situ* field test specialists for geophysical and geotechnical measurements, specialists in obtaining field samples for laboratory testing, and others. In addition, the data of interest generally are referenced to a geographic location. Consequently, the effort must involve specialists in spatial mapping and location technologies; e.g., geographic information systems (GIS).

A work group is currently being established for the purpose of developing an expansible data dictionary. The work group is constituted of representatives of data providers, such as Caltrans and other state DOTs, CGS, U. S. Geological Survey (USGS), Pacific Gas and Electric Company (PG&E), Federal Highway Administration (FHWA). The work group should have participation from professional associations such as American Society for Testing and Materials (ASTM), AAS, non-profit organizations such as Petrotechnical Open Software Corporation (POSC), and should include specialists in GIS technology. The work group should be organized around subgroups necessary to address the range of specific types of geotechnical data to be contained in databases that are to be linked to the web-based virtual system. Subgroups should include geologic subjective non-parametric data, *in situ* geotechnical test data, *in situ* geophysical test data, field sampling and laboratory test data, and spatial and site location information. In order to insure that adequate communication takes place across data types it is recommended that the subgroups all be part of the larger work group.

Data Quality Issues

Data quality is a key issue that must be addressed as a part of the curation of data. Responsibility for data quality must rest with data providers and those who collect data that may become part of a data provider's database. Quality assurance is viewed not so much achieved through process in the form of QA procedures or guidelines as through instilling a culture of quality data reporting. Some assurance of quality can be attained through data checking procedures that can be easily implemented by data providers. Even greater quality assurance can be attained by implementing procedures for data entry that avoid handling of the data recording by multiple persons along the pathway to archiving. The most effective procedure would be to implement mechanisms to enter data into the archive directly as it is obtained in the field. The overriding need is to stimulate those who collect data to think about quality reporting as part of routine data acquisition.

Mechanisms and procedures for assuring data quality necessarily are different for new data and for legacy data (Nigbor et al., 2002). For new data, quality assurance is highly related to the scope and content of the data dictionary as well as to the mechanisms and procedures employed for entering data into a database. For legacy data little can be done along these lines, as the data already exist and are held in a particular format. For these data the primary quality assurance need is to avoid data transfer error in the process of capturing the data in electronic format. It is believed to be possible however, to provide some measure of the quality of legacy data by assessing the degree to which the information available meets the data dictionary requirements for new data entry. By implementing this process, legacy data could be rated according to a measure of quality.

Data quality should be an integral consideration for the development of a comprehensive data dictionary standard. The NGES data dictionary takes data quality into account and is considered a good starting point for the development of a data dictionary standard suitable for the envisioned web-based virtual system. It is believed likely that once a data dictionary standard is in place it will be generally accepted and used by most organizations and contractors. Nevertheless, when data are acquired through specific contractual agreements use of the standard should be required. To the extent achievable procedures should be developed and used to enter data electronically as they are obtained in the field.

In order to flag the quality of legacy data a procedure should be developed for rating the data based on the degree to which the available documentation meets the data dictionary standard.

Development of a Pilot Virtual Geotechnical Data Center System for Caltrans and CGS Geotechnical Databases

This section describes actions for development of a Pilot Virtual Geotechnical Data Center System for CGS, USGS, PG&E and Caltrans geotechnical data sets, which could be expanded in the future to incorporate the broad range of geotechnical data from other agencies, academia, and industry. A schematic of how the CGS or Caltrans data might be captured and structured as an element of the pilot virtual system is shown in Figure 7.

A work group is currently being formed to identify and define optimal information architecture (including archiving) for the virtual system hub, considering the primary agencies' needs, currently available technologies, probable future technological developments, future expansion to include other geotechnical databases, and potential integration with Network for Earthquake Engineering Simulation (NEES) Program. The focus is on implementing the information harvesting architecture recommended by the workshop participants. In addition to defining the basic system architecture that meets the functional requirements of a web-based system, specific responsibilities for maintaining the data dictionaries, the structures for holding and disseminating metadata, the process for developing and structure for holding data indexes, and the responsibility for developing and maintaining data translators should be identified. It is anticipated that these elements would be to some degree unique to each data provider.

One of the primary steps involved in creating the pilot system is to develop a basic system integration plan for each agency that could be expandable to a larger web-based system serving multiple data sets. Of equal importance, data indexes and translator methods and technologies required by each of the primary data providers must be clearly defined. Metadata requirements for data providers to participate in the virtual system would also be defined. A phased implementation plan and organizational structure for each agency to participate in the virtual system should be developed.

Structure and Method for Handling Metadata and Data Exchange

In order to create a virtual system, the structure and method for handling metadata, non-metadata and data exchange needs to be addressed. The structure and methods will be dependent on the user scenarios. The translators and indexes tend to be application-specific, and may be unique to each data provider, though a number of different providers could share a given index.

The structure of the metadata and the data disseminated through the hub must be less specialized, in order to facilitate the implementation of the virtual system. The more portable the technology developed, the easier it would be to expand the system in the future to include other participants.

In order to define steps involved in determining the structure and methods for handling metadata and data exchange standards, it is necessary to define protocols and architectures for accessing metadata and disseminating non-metadata produced by each data provider. Protocol and architecture refers to data processing and platform-specific functionality to be used for data translation and exchange. These protocols and architectures might be unique to each agency, depending on the agencies' needs, currently available technologies, probable future technological developments, and available maintenance resources. For instance, government agencies are often restricted in terms of allowable software, use of newer markup languages, and server-side programming (such as Active Server Pages, ASP) in regards to what can be deployed over the internet. Applicable solutions for data exchange could be determined based on available resources, personnel, and protocols appropriate for the project.

Setting up metadata and non-metadata generation schemes at each agency can be handled in a number of ways. Software programs may be written specifically to translate and extract metadata and data from the agencies' native databases or data archives into a form which can be queried by the virtual system. The recommended amount of data, or rather non-metadata that would be contained within each metadata file depends primarily on how large a given data set is. If the data set for a single borehole or geotechnical investigation site, for instance, is comprised of text information only, then the metadata files would be very small and could contain all of the non-metadata within a given file.

The metadata and non-metadata files should be made accessible through an index of the agencies' currently available files connected to or residing on the virtual system. Recommendations for where the metadata, non-metadata and indexes should reside can be resolved during development of the pilot system. The translators could reside with and be maintained by data providers or they could reside at the system hub and be maintained there depending on their function.

A small work group is being formed to develop metadata structures and translators specific to each agency, aiming toward less specialized data exchange methods for participation in the system hub. The work group consists of persons from the data provider organizations and an IT specialists who have experience with metadata and data exchange standards development. In addition to the above-mentioned technical needs, the work group will also address possible solutions to allow geotechnical data to be viewed on-the-fly as graphics from the system hub.

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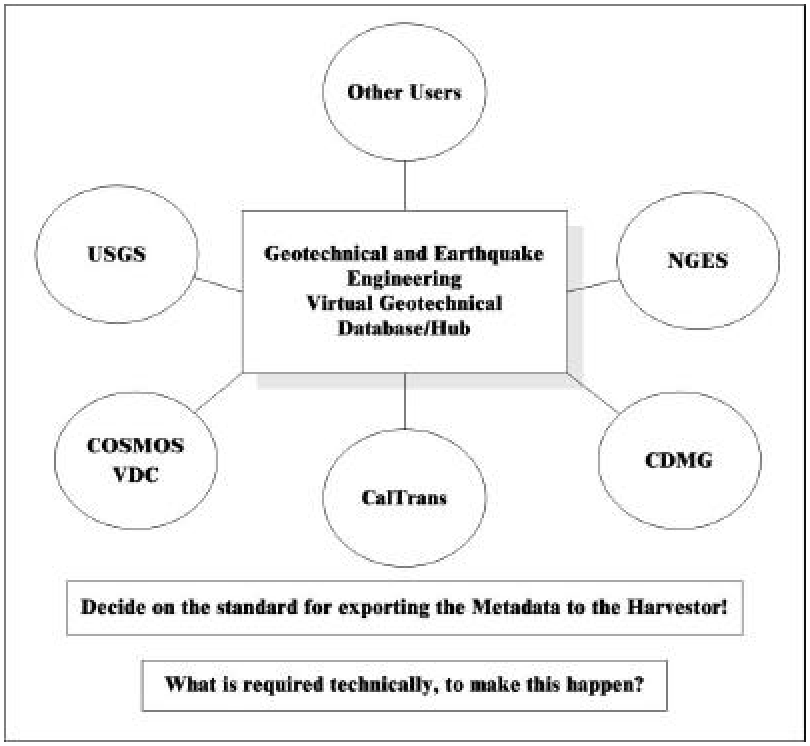

Figure 1. Generalized concept of a web-based virtual data dissemination center.

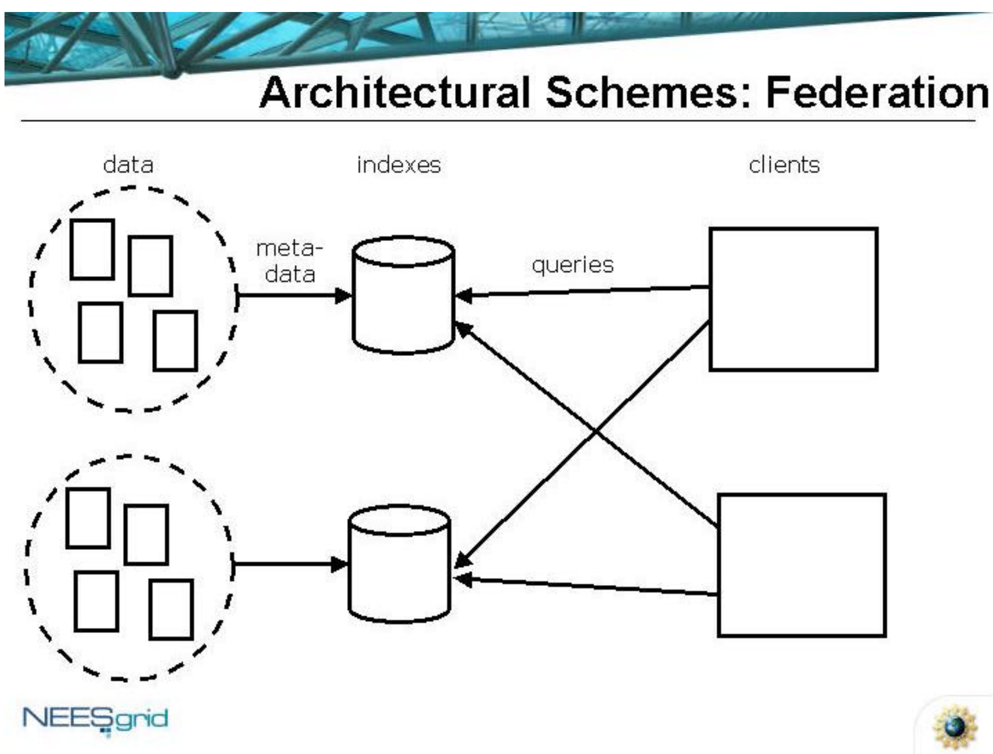

Figure 2. Information architectural scheme: "Federation" (Joe Futrelle, Stepp et al., 2002).

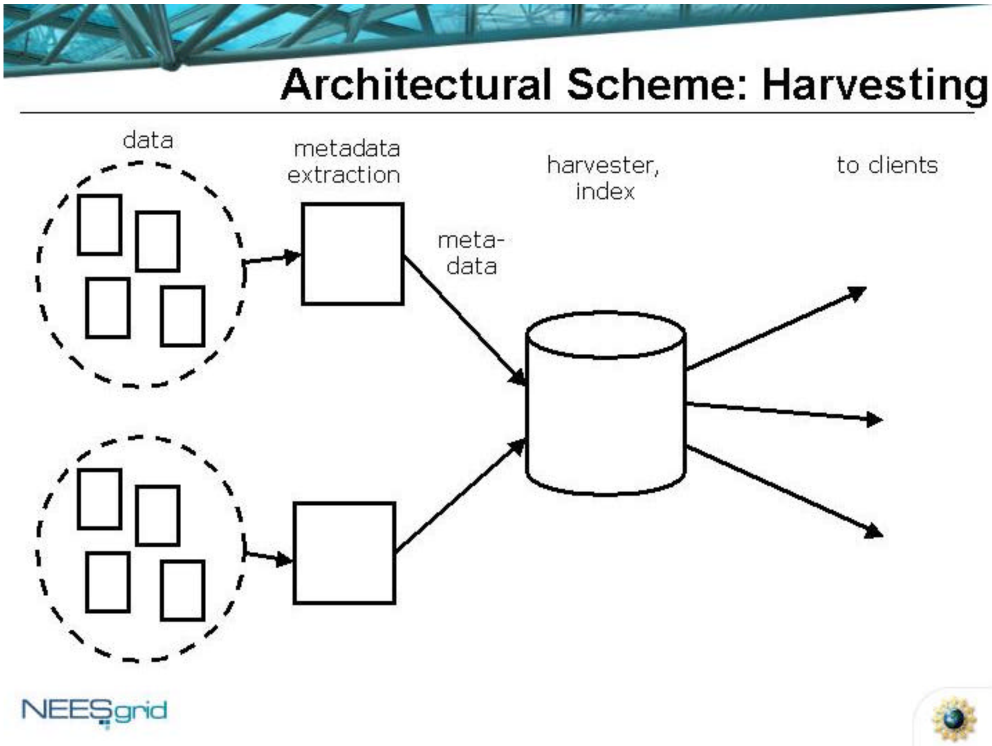

Figure 3. Information architectural scheme: “Harvesting” (Joe Futrelle, Stepp et al., 2002).

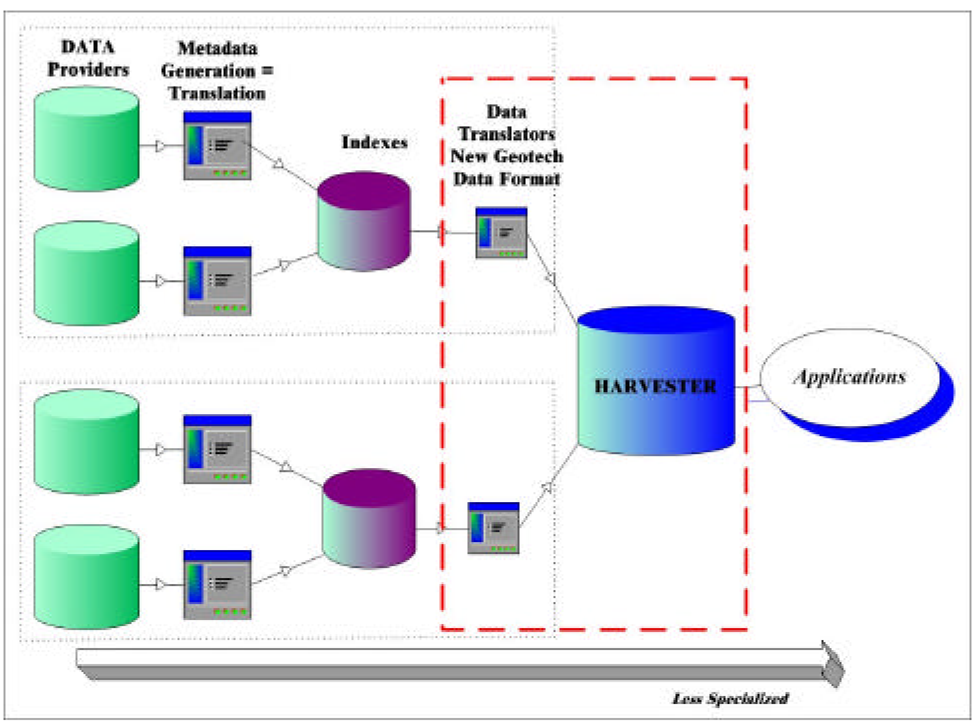

Figure 4. Illustration of a harvesting architecture for a web-based virtual geotechnical data center for four data providers.

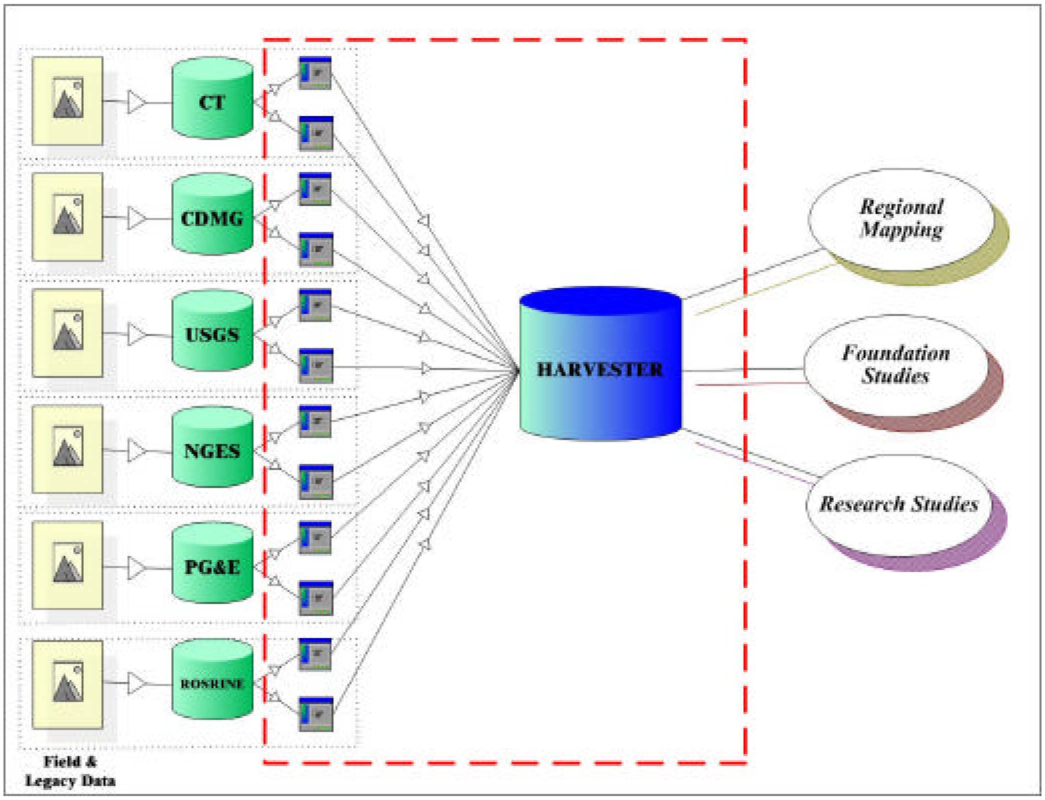

Figure 5. Basic elements of an extensible web-based virtual geotechnical data center.

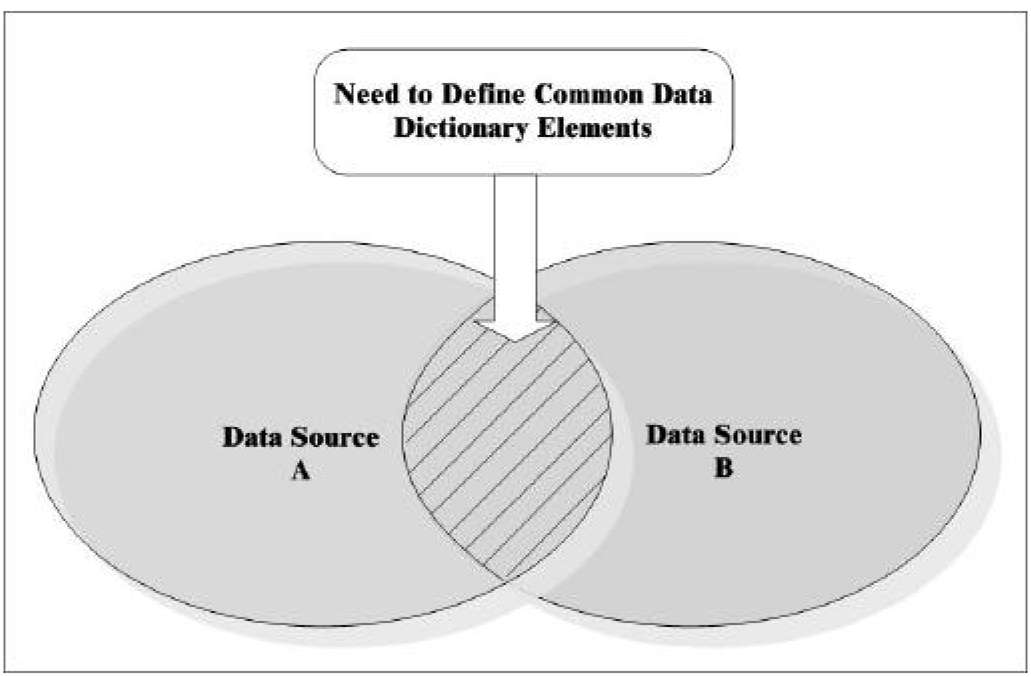

Figure 6. Schematic depiction of an approach for developing a data dictionary standard.

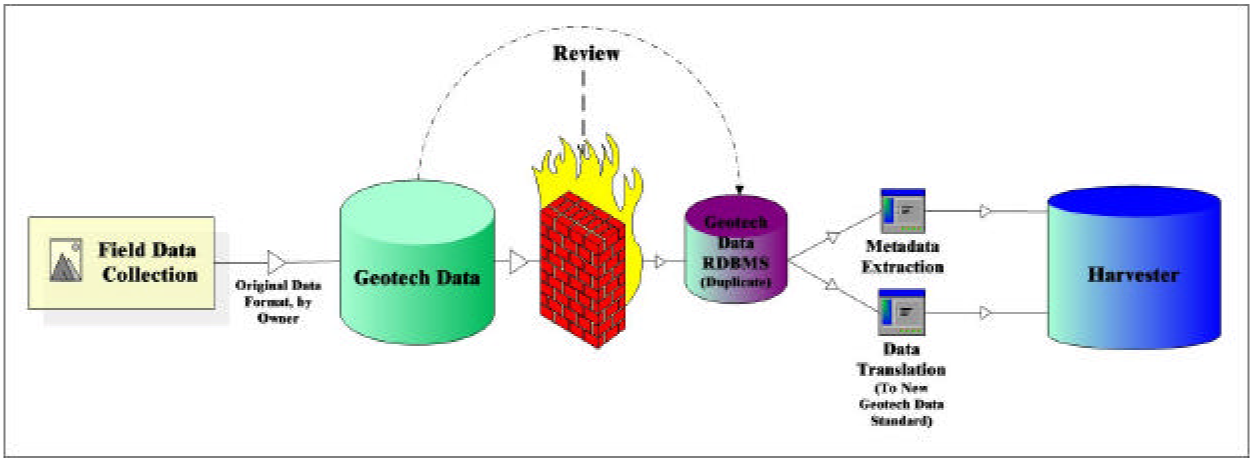

Figure 7. Schematic showing how CGS or Caltrans data could be captured and a structure for participation in a virtual geotechnical database system.